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DETERMINING THE UNDRAINED SHEAR STRENGTH OF OVERCONSOLIDATED CLAY

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New York State Department of Transportation

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ABSTRACT

Part I of the paper presents a general discussion of the causes of overconsolidation in a clay deposit and methods of identifying overconsolidation. Some background information is given on methods of soil testing and sources of error, and the effects of overconsolidation on soil behavior.

Part II of the paper deals with the Cam-Clay Prediction of Undrained Shear Strength, as presented by P.W. Mayne (1980). The Cam-Clay method estimates the undrained shear strength of a given overconsolidated cohesive soil as a function of the undrained shear strength when normally consolidated, the degree of consolidation, and the consolidation indices. The method is briefly explained and several equations are presented for use in design.

Part III is a collection and study of data from selected projects in New York State. Triaxial compression test data from a sample of Lockport Clay is used to compare the undrained shear strengths obtained from the Cam-Clay Method and from the Soil Mechanics Bureau (SMB) Procedure. A step by step procedure to estimate the Overconsolidation Ratio (OCR) and the undrained shear strength by the Cam-Clay method is given to show the proper procedure to obtain the necessary parameters.

Appendix A contains the plots of C/P vs. OCR from which Λ was obtained for this study. Appendix B contains plots of the results of regression analyses on (C/P)nc vs. Plasticity Index and vs. Moisture Content from data collected for the study. Appendix C is a plot showing the influence of plasticity index on the effective angle of internal friction, and Appendix D is a summary of all the data collected and contains frequency histograms for each parameter studied.

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Part II of the paper deals with the Cam-Clay Prediction of Undrained Shear Strength, as propounded by C.P. Wroth (1980). The Cam-Clay method estimates the undrained shear strength of a given overconsolidated cohesive soil as a function of the undrained shear strength when normally consolidated, the degree of consolidation, and the consolidation indices. The method is briefly explained and several equations are presented for use in design.

Part III is a collection and study of data from selected projects in New York State. Triaxial compression test data from a sample of Lexington clay is used to compare the undrained shear strengths obtained from the Cam-Clay Method and from the Soil Mechanics Bureau (SMB) procedure. A step by step procedure to estimate the Overconsolidation Ratio (OCR) and the undrained shear strength by the Cam-Clay method is given to show the proper procedure to obtain the necessary parameters.

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PART I

PROCESSES THAT CAUSE OVERCONSOLIDATION

It is well known that the degree of overconsolidation greatly affects the strength and compressibility characteristics of a cohesive soil deposit. As a rule, the greater the degree of overconsolidation, the greater the strength and the lesser the compressibility. Overconsolidation is defined as the condition by which a soil has been subjected to a higher effective overburden pressure in the past than the soil is presently experiencing. The Overconsolidation Ratio (OCR) is defined as the maximum past effective pressure, Pp, divided by the existing effective overburden pressure, Po.

One of the most pertinent factors that must be considered in the evaluation of soil strength is the geological stress history of the soil deposit. By knowing the history and the processes that may have caused overconsolidation, the design engineer will have a better understanding of how the maximum past pressure, Pp, should vary with depth, and will therefore be able to identify values that have deviated from the "expected" values due to sampling and testing procedures or natural events.

There are many physical processes that can cause overconsolidation in a soil. The first process is a reduction in total stress, such as removing overburden by erosion or excavation, removal of past structures, or the withdrawal of a glacier. The second process is a change in pore water pressure. A rise in the water table or an introduction of an artesian pressure will decrease the effective unit weight of the soil, and will therefore cause the effective overburden pressure to decrease. Conversely,

the effective overburden pressure will increase if the effective unit weight of the soil increases, such as by a lowering of the water table, or by desiccation due to drying or capillarity.

Overconsolidation processes are not limited to changes in vertical effective stress, however. T.C. Kenney (1968) notes that any change in the soil that will cause increased shear resistance between the individual soil particles will cause increased resistance of the soil to compression. Leonards and Altshcaeffl (1964), and Bjerrum (1967) have shown that changes in soil structure due to secondary consolidation result in a "quasi-preconsolidation." Overconsolidation can also be influenced by environmental changes in the soil, such as changes in pH, temperature, and salt concentration (Lambe, 1958a and b), or by chemical alterations in the soil, such as weathering, precipitation, cementing agents, and ion exchange (Bjerrum, 1967).

Although special terms are suggested to describe these causes of overconsolidation, it is difficult to establish for most clay deposits exactly when a particular cause would apply. Also, the maximum past pressure represents a stress that separates small strain "elastic" behavior from large strains accompanied by plastic deformation during one-dimensional compression. Because of this, it is appropriate for design purposes to treat the "breaking point" in the e-log P curve as the maximum past pressure regardless of the physical cause of the overconsolidation.

FACTORS CONTROLLING STRENGTH AND METHODS OF TESTING

In reality, the shearing resistance or strength of a soil depends on many factors. A complete equation might be of the form:

$$\text{Shearing Resistance} = F\,(e,\ \phi',\ C,\ \sigma',\ c',\ H,\ T,\ \epsilon,\ \dot{\epsilon},\ S)$$

in which e is the void ratio, ϕ' is the friction angle in terms of effective stress, C is the composition, σ' is the intergranular pressure, c' is the cohesion, H is the stress history, T is the temperature, ϵ is the strain, $\dot{\epsilon}$ is the strain rate, and S is the structure. All of the parameters in the above equation may not be independent, however, and their quantitative functional forms are not well known. Consequently, values of c' and ϕ' are determined using a specified test type, drainage condition, rate of loading, and stress history. As a result, a variety of types of "friction angle" and "cohesion" have been defined, including parameters for total stress, effective stress, drained strength, undrained strength, peak strength, and residual strength.

Several sources of error exist with each test and specimen. Two very significant sources of error are 1) the sample disturbance due to the stress release when the sample is removed from the ground and 2) the sample disturbance due to the shear distortion of the original arrangement of the soil structure during sampling (Berre and Bjerrum, 1973). Sample disturbance due to shear distortion proves to be especially detrimental to soft clays with a brittle structure. Sample disturbance due to stress release increases with depth, and can result in errors of up to 50% at depths below 20-30 feet (Ladd and Lambe, 1968). Disturbance due to sampling is more detrimental to clay samples of low plasticity than to samples of plastic clay or cemented clay. Even though the in-situ stresses can be restored if the sample, prior to its testing, is reconsolidated in the laboratory at the same stress it carried in the field, some effects of disturbance remain. Sample disturbance is also introduced during trimming

and handling the sample in preparation for testing.

The undrained shear strength of a cohesive soil along a potential failure plane also varies due to the inherent strength anisotropy of clays and the reorientation of the principal planes during shear (Hansen and Gibson, 1948). As the PI decreases or sensitivity increases, the anisotropy effects on shear strength increase.

Another factor influencing the strength of the soil specimen is the rate of strain at which the sample is brought to failure. Generally, with each log cycle decrease in the rate of strain there is an approximately 10% decrease in the undrained shear strength (Ladd and Foott, 1974). That is, as the rate of strain is slowed, creep effects become more significant and therefore cause a reduction in shear strength. Or conversely, as the rate of strain increases, viscous resistances become more significant and cause an increase in shear resistance. However, Gemme (1985) shows that for the majority of New York State soils with an average PI of 15-20, the strength variation due to strain rate effects is less than 5% and can be neglected.

There are three basic types of triaxial tests used to determine the shear strength of a specimen: Unconsolidated Undrained, Consolidated Undrained, and Consolidated Drained.

1. Unconsolidated Undrained (UU) test results tend to be lower than the in-situ strength for soft clay with low plasticity and organic silt because of a reduction in effective stress due to sampling disturbance,

swelling of the sample, or expansion due to the escape of gases. The average time to perform UU tests is very short because the sample does not need to be consolidated overnight, as is necessary in other testing. This characteristic may be useful at times when strength parameters are needed and there is not enough time to run a Consolidated Undrained test.

2. The Consolidated Undrained (CU) test is the most widely used strength test for the design of embankment foundations. The CU test, however, overestimates the shear strength by about 15% (Ladd and Lambe, 1968), but after correcting for the effects of sampling disturbance, the results are fairly accurate.

3. Consolidated Drained (CD) tests are used in the design of cut slope and natural slope stability treatments. The advantage of a CD test is that the results are usually not significantly affected by sample disturbance. The disadvantage of the CD test is that it is difficult to conduct in the lab. To insure that there is no build up of pore pressure in slow draining samples during shear, the rate of strain must be very slow, with the time to failure ranging from one day to several weeks. With test durations of these lengths, problems may develop with the testing equipment, such as leaks in seals, valves, and the membrane surrounding the sample. An alternative method to obtain the strength parameters in terms of effective stress is to use the CU test, measure the pore pressures, and then subtract out the pore pressure effects. The effective strength parameter from the CU test is slightly higher than from the CD test, but the error introduced is insignificant from the practical point of view.

The most conventional method of measuring the shear strength along the vertical plane of a cohesive soil deposit is by the field Vane Shear test. Because the test is performed in-situ, sampling disturbance is minimized. If the test is performed correctly and the soil is uniform and without shells or pebbles, the measured strengths will generally be reliable. However, because the Vane Shear test is a relatively rapid test, an error can be introduced in soils of high plasticity due to the rapid strain rate. This error is compensated for by using the Bjerrum correction factor (Bjerrum, 1972). The Bjerrum correction is based on PI, with the correction factor decreasing with increasing PI. However, most New York State clay-type soils have PI's between 10 and 30, and the correction factor for this range is approximately equal to one. Therefore, the correction is usually neglected for clay-type soils in New York State.

IDENTIFICATION OF OVERCONSOLIDATION

There are a few qualitative methods of identifying an overconsolidated soil deposit. One method is based on the liquidity index (moisture content minus plastic limit divided by plasticity index), initially proposed by Terzaghi. A heavily overconsolidated soil generally has a liquidity index equal to or close to 0, whereas the liquidity index of a normally consolidated or slightly overconsolidated insensitive soil is about 0.5 to 1.0. For sensitive soft clays, the liquidity index is generally greater than 1.0.

A second qualitative method of determining overconsolidation is to observe the characteristics of the soil deposit as they vary with depth. In a normally consolidated clay deposit, shear strength tends to increase with depth, but moisture content decreases with depth. In an overconsolidated

clay deposit, shear strength either decreases then increases with depth or does not vary significantly with depth.

Through interpretation of laboratory test results, the degree of overconsolidation can be determined more exactly. From a plot of void ratio vs. log P (from consolidation tests), the maximum past overburden stress Pp can be estimated. If good quality samples are obtained and there is a sharp break in the curve, the Casagrande construction (Figure 1) may be used as a good approximation. If the curve does not break sharply, or more accuracy is desired, then the Janbu (Janbu, 1965) or the Schmertmann (Leonards, 1962) procedures may be used. Pp can also be determined from the failure envelope of Mohr Circles generated from triaxial test results (Figure 2), or from the OCR vs. strength ratio curve as shown on Page 3-16 of the Soil Mechanics Bureau Design Manual (SMB Procedure).

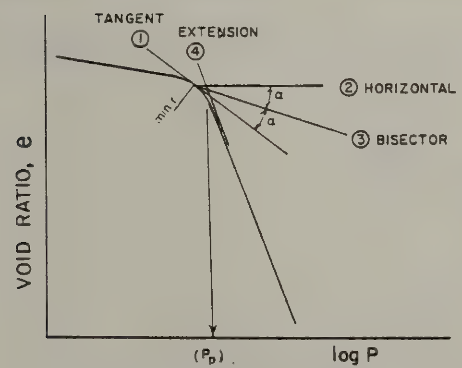


FIGURE 1: CASAGRANDE CONSTRUCTION
(CASAGRANDE, 1936)

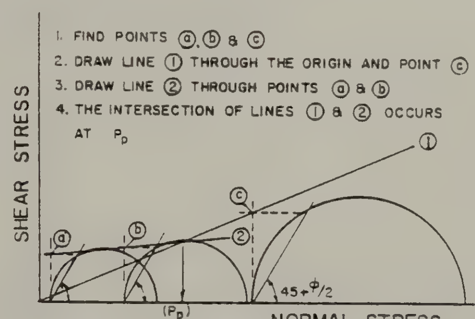


FIGURE 2: DETERMINING P_p FROM TRIAXIAL TEST RESULTS
(BASED ON NAVFAC DM 7.1, 1982)

EFFECTS OF OVERCONSOLIDATION ON SOIL BEHAVIOR

The stress-strain behavior for normally consolidated (NC) soil and overconsolidated (OC) soil for CD or CU triaxial compression tests is as shown in Figure 3. Notice that the OC sample behaves more or less like a brittle material, has a higher peak stress at a lower strain, and has a higher modulus than a NC sample at the same confining pressure.

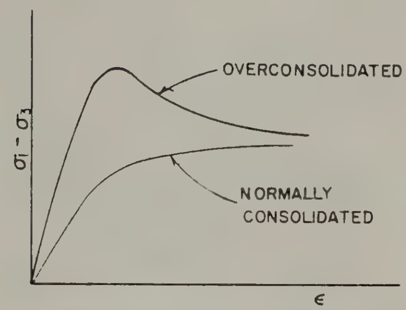


FIGURE 3: TYPICAL STRESS-STRAIN RELATIONSHIP

Figure 4 shows the change in volume vs. strain relationship for NC and OC samples in a CD test. During shear, the NC sample consolidates, while the OC sample first consolidates for a small strain, then dilates.

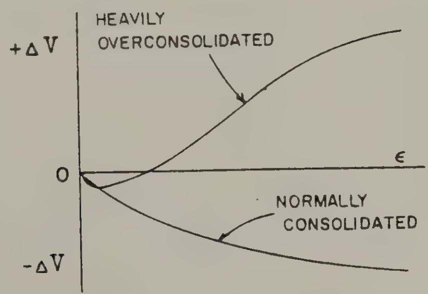


FIGURE 4: TYPICAL VOLUME-STRAIN RELATIONSHIP
(BASED ON HENKEL, 1956)

A similar relationship exists for change in pore water pressure vs. strain in a CU test as shown in Figure 5. On a NC specimen, positive pore water pressures develop and the water has a tendency to be forced out during shear. As an OC specimen is sheared in a CU test, positive pore pressures develop during the early stage of shearing, then as further strain takes place, negative pore pressures develop.

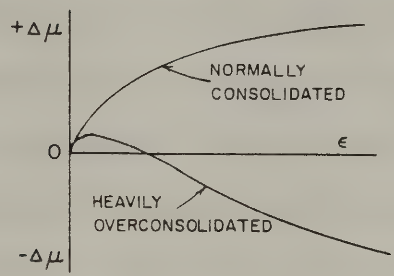


FIGURE 5: TYPICAL PORE PRESSURE VS. STRAIN RELATIONSHIP
(BASED ON HENKEL, 1956)

PART II

NORMALIZED BEHAVIOR

Great progress has been made in the scientific study of the overconsolidated undrained strength of cohesive soils during the past three decades. It has been recognized that the majority of clay samples with the same OCR but different consolidation pressures (and therefore different maximum past pressures; recall OCR = Pp/Po) have similar undrained strength and stress-strain characteristics when the shear strength is divided by its corresponding consolidation pressure (Ladd and Foott, 1974). This relationship, referred to as "normalized" behavior, is important in the Cam-Clay Predictions of Undrained Shear Strength (P. W. Mayne, 1980).

A plot of normalized shear strength vs. OCR on a semi-log or log-log scale can be used to determine the undrained shear strength of a given deposit consolidated at various degrees of overconsolidation. However, soils with a high degree of structure, such as quick clays and naturally cemented clays, will not have a normalized behavior. During consolidation, the soil structure of these clays is re-formed, so the normalized behavior relationships do not apply (Ladd and Foott, 1974).

CAM-CLAY PREDICTIONS OF UNDRAINED SHEAR STRENGTH

Since the strength of a cohesive soil is very much dependent on its degree of overconsolidation, it would be to the designer's advantage to represent the undrained shear strength in such a way that the effects of overconsolidation can be directly considered in the analysis. One such method of determining the undrained shear strength is by the Cam-Clay Predictions of Undrained Shear Strength, as presented by P.W. Mayne (1980).

The Cam-Clay concept (Roscoe and Burland, 1968, and Schofield and Wroth, 1968) is a plastic theory based on the correlation between strength, effective stress, and water content. The method describes the shear strength at the critical state or residual condition, which represents deformation under the condition of constant volume and constant fabric. Although the Cam-Clay prediction of undrained shear strength was originally developed for isotropically consolidated soils, it has been shown that the method can be successfully used to estimate the undrained shear strength for anisotropically consolidated clays and silts (Mayne, 1980). In this study, application of the method will only be made to triaxial compression test results. Strength anisotropy due to stress anisotropy and reorientation of the principle stresses during shearing is not considered.

The distinct advantage of the method is that only two soil constants are needed to represent the undrained shear strength for any degree of overconsolidation: effective friction angle ϕ', and the critical state pore water pressure parameter Λ. Λ is defined as the slope of a linear relationship between (C/Po)oc vs. OCR on a log-log scale and may be expressed as follows:

$$\Lambda = \frac{\log [(C/Po)oc] - \log [(C/Po)nc]}{\log [OCR]} \quad \ldots\ldots\ldots.(1)$$

Where Po is defined as the effective overburden pressure and C is the undrained shear strength; the subscripts 'oc' and 'nc' represent overconsolidated and normally consolidated, respectively.

Rearranging Equation (1) yields:

$$\frac{(C/Po)oc}{(C/Po)nc} = OCR^{\Lambda} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

Theoretically, the parameter Λ is equal to 1-(Csi/Cci), in which Csi & Cci are the isotropic swelling and compression indices respectively. As a close approximation, Atkinson and Bransby (1978) have proposed:

$$\Lambda = 1-(Cs/Cc) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where Cs and Cc are the swelling and compression indices respectively, from conventional consolidation tests.

Mayne suggests that it is better to determine Λ from Equation (1) or Equation (5) based on CU or CD triaxial results rather than from Equation (3), which uses conventional consolidation results, because there are problems in determining Cs in consolidation tests (Mayne, 1980). Generally, there is not much attention given to defining a value or range of values for Cs during testing, and as can be seen from results of a typical consolidation test (Figure 6), the plot of void ratio vs. log effective stress for Cs is actually a non-linear relationship.

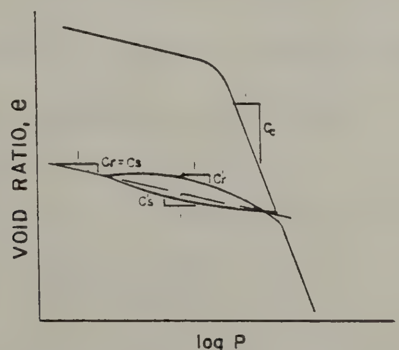


FIGURE 6: TYPICAL PLOT OF
CONSOLIDATION TEST RESULTS

However, if the results of consolidation testing are to be to used to obtain Λ, adequate approximations can be made using an average slope for Cs. The error introduced by estimating the slope is small and Λ itself does not have a substantial influence on the shear strength. Therefore, it is not necessary to determine Cs precisely.

A plot of log (C/P) vs. log OCR will vary from a horizontal line at $\Lambda = 0$ to a line at 45 degrees for $\Lambda = 1$. Λ theoretically lies between 0 and 1, but the actual physical meaning of these boundaries is not clear. For example, for Λ to be 1, according to Equation (2), (Cs/Cc) would have to be 0. For this to be true, either Cs would have to be 0, or Cc would have to be infinity. On a plot of e-log P, a rebound curve with Cs of 0 would be a horizontal line, and a curve with Cc of infinity would be a vertical line. For Λ to be 0, Cs would have to equal Cc, a condition which indicates that the rebound curve rebounds back along the virgin compression curve. Soils with properties such as these do not exist. It is suggested, then, that the actual values of Λ will lie in a range somewhere within the theoretical limits.

By comparing a wide variety of test results reported by various researchers from countries all over the world, Mayne reconfirmed the validity of the Cam-Clay theory and concluded that the undrained shear strength to effective overburden pressure ratio for NC soils can be represented in terms of the effective shear strength parameter and Λ by:

$$\left(\frac{C}{Po}\right)_{nc} = \frac{M}{2} e^{-\Lambda} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

where $M = \frac{(6 \sin \phi')}{(3 - \sin \phi')}$

and $e = 2.718$

Thus: $$\Lambda = \frac{\ln [(2/M)\ (C/Po)]}{\ln (OCR)-1} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

From Equation (5) the in-situ OCR can be back calculated for specific confining stress levels:

$$OCR = \left(\left(\frac{2}{M} \right) \left(\frac{C}{Po} \right)^{\Lambda} \right)^{1/\Lambda} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

To predict the effect of OCR on the undrained strength of clays, solve Equation (5) for (C/Po):

$$(C/Po) = \frac{(3 \sin \phi')}{(3-\sin \phi')} \left(e^{-1}\ OCR \right)^{\Lambda} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

There is a slight discrepancy between the predicted, theoretical values of shear strength by the Cam-Clay model and the actual, experimental values. To compensate for this variation, Mayne (1980) suggests that an "attraction" of 10% be added to the values of Po in Equations (1) through (7).

PART III

ANALYSIS OF DATA FROM SELECTED PROJECTS IN NEW YORK STATE

The study of New York State cohesive soils includes test data gathered from projects in three regions: Region 1, Region 5, and Region 11. From Region 1, information was gathered from 61 CU tests on undisturbed samples taken from several projects including: Alternate Route 7, projects in Albany-Couse, the South Mall, projects in Fort Ann-Whitehall, and Route 9W Ravena-Becker. From Region 5, 76 CU tests on undisturbed samples taken on the Lockport Expressway were studied. In addition, information provided by Woodward-Clyde Consultants, consultants for Westway, was used for Region 11. The projects chosen for the study are arbitrary and do not represent New York State in any statistical pattern.

From the Mohr circle plots of the results of CU triaxial compression tests, (C/P) ratios were read for OCR's of 1 (NC), 2, 5, 10, and 20. Log (C/P) was then plotted vs. log OCR (Appendix A) for each project group. From these plots, Λ (the slope of log (C/P) vs. log OCR) was read and then plotted vs. Plasticity Index (PI) and Moisture Content (MC) (Appendix B).

The scatter in the results is most likely attributable to the effects of sample disturbance, variations in sample homogeneity and errors in obtaining values of Pp from the test results. Region 1 clays have a range of Λ of 0.4 to 0.7 (Figure B1-B5), Lockport clays Λ range from 0.65 to 0.9 (Figure B6), and Λ for Westway clays range from 0.8 to 0.92 (Figure B7).

Also in Appendix B are the results of sidelight studies examining the relationships between (C/P)nc and PI and between (C/P)nc and MC. There is literature to support two variations in the relationship between (C/P)nc and PI; one variation by Skempton (1957) and another by Osterman (1959).

For normally consolidated sensitive clays and plastic clays, Skempton established the following empirical correlation based on field vane tests:

$$\left(\frac{C}{P}\right)_{nc} = 0.11 + 0.0037\,(PI) \qquad (8)$$

Bjerrum's study of embankments on soft ground foundations that failed under undrained conditions indicated that a reduction factor, r, should be applied to the results of undrained laboratory tests or field vane tests on clays of high plasticity. The reduction factor may be approximated by the following equation:

$$r = 1.0 - 0.5 \log\left(\frac{(PI)}{20}\right) \text{ for } PI \geq 20 \qquad (9)$$

Thus, the in-situ undrained strength ratio for NC clays can be estimated from Equation (8) and appropriately corrected by Equation (9). Skempton's equation when plotted shows (C/P)nc increasing with increasing PI. However, for the majority of New York State soils generally characterized as insensitive clays or clayey silts of low plasticity, (C/P)nc decreases with increasing PI. This trend is in agreement with Osterman's finding.

By examining the plots in Appendix B, the two variations in the relationship between (C/P)nc and PI can be seen. Generally, it appears that for Lockport

clay, (C/P)nc decreases with increasing PI. The Westway clay curve generally shows (C/P)nc increasing with increasing PI. For Region 1 soils, however, it appears that some clays have (C/P)nc decreasing with increasing PI and other clays appear to have (C/P)nc increasing with increasing PI. It is interesting to note that (C/P)nc vs. MC relationships shown in Appendix B correlate almost as well or better than (C/P)nc vs. PI relationships.

It should be noted that the data for many of the plots shown in Appendix B is limited and therefore the level of confidence in the correlation coefficients is not very high. However, the plots do show general trends and indicate the range of values that can be expected at each of the project locations. Figure D1 in Appendix D shows Frequency Histograms for Moisture Content, Plasticity Index, (C/P)nc, and Λ for all the data collected in this study.

COMPARISON OF CAM-CLAY METHOD TO SOIL MECHANICS BUREAU PROCEDURE

An example problem demonstrating the calculation of the undrained shear strength by the Cam-Clay method using data from triaxial compression shear tests follows. The results are compared to the undrained shear strength obtained from the design chart on Page 3-16 of the Soil Mechanics Bureau (SMB) Design Manual. The soil data used for the comparison is from the Lockport Expressway, Hole No. 534B. The soil is described as clay and silty clay, with moisture contents ranging from 33 to 48, PI ranging from 15 to 25, and (C/P)nc about 0.22.

COMPARISON OF SHEAR STRENGTHS OBTAINED BY CAM-CLAY METHOD AND SMB PROCEDURE

1. The parameter Λ can be obtained from: log C/P vs. log OCR curves (Figures A6-A9); Equation (3); Figure B6 using PI = 22; or use the mean value of 0.75 found in this study (Figure D1). Use value of Λ = 0.75 for this comparison.

2. Assuming γ' = 58 pcf (Cam-Clay method applies to saturated soils) and knowing Pp from testing, plot Po and Pp curves (Figure 7).

3. Obtain Ø' from Figure C1 (Appendix C).

4. Using Equation (7):

$$\frac{C}{Po} = \frac{3 \sin \phi'}{3-\sin \phi'} (e^{-1}\ OCR)^{\Lambda}$$

$$= \frac{3 \sin \phi'}{3-\sin \phi'} \left(\frac{OCR}{e} \right)^{.75}$$

5. Calculate the undrained shear strength C by setting up Table 1.

6. Plot C vs. depth (Figure 8).

TABLE 1: CALCULATION OF SHEAR STRENGTH VS. DEPTH BY CAM-CLAY METHOD

DEPTH	OCR	Ø' (a)	$\frac{C}{Po}$	1.1 Po (b)	C
8	11.6	24	1.40	510	712
10	7.8	24	1.04	638	661
12	5.8	24	0.83	766	635
14	4.6	23	0.67	893	595
16	3.7	23	0.57	1021	578
18	3.2	23	0.51	1148	583
20	2.7	23	0.45	1276	570
22	2.4	24	0.43	1404	601
24	2.2	25	0.42	1531	643
26	2.1	25	0.40	1659	672
28	2.0	25	0.39	1786	698

(a) Based on variations of PI vs. Depth.

(b) Correction Factor as described on Page 14.

TABLE 2: CALCULATION OF SHEAR STRENGTH BY SMB PROCEDURE

DEPTH	OCR	$\frac{Su}{Cu}$	$\left(\frac{C}{P}\right)_{nc}$ (c)	Su @ $\left(\frac{C}{P}\right)_{nc}$	Su by SMB
8	11.6	.185	.24	111	602
10	7.8	.25	.24	139	557
12	5.8	.30	.21	146	487
14	4.6	.35	.21	170	487
16	3.7	.41	.21	195	475
18	3.2	.46	.21	219	477
20	2.7	.51	.22	255	500
22	2.4	.55	.22	281	510
24	2.2	.58	.22	306	528
26	2.1	.60	.22	332	553
28	2.0	.615	.22	357	581

(c) Based on shear strengths obtained from triaxial testing.

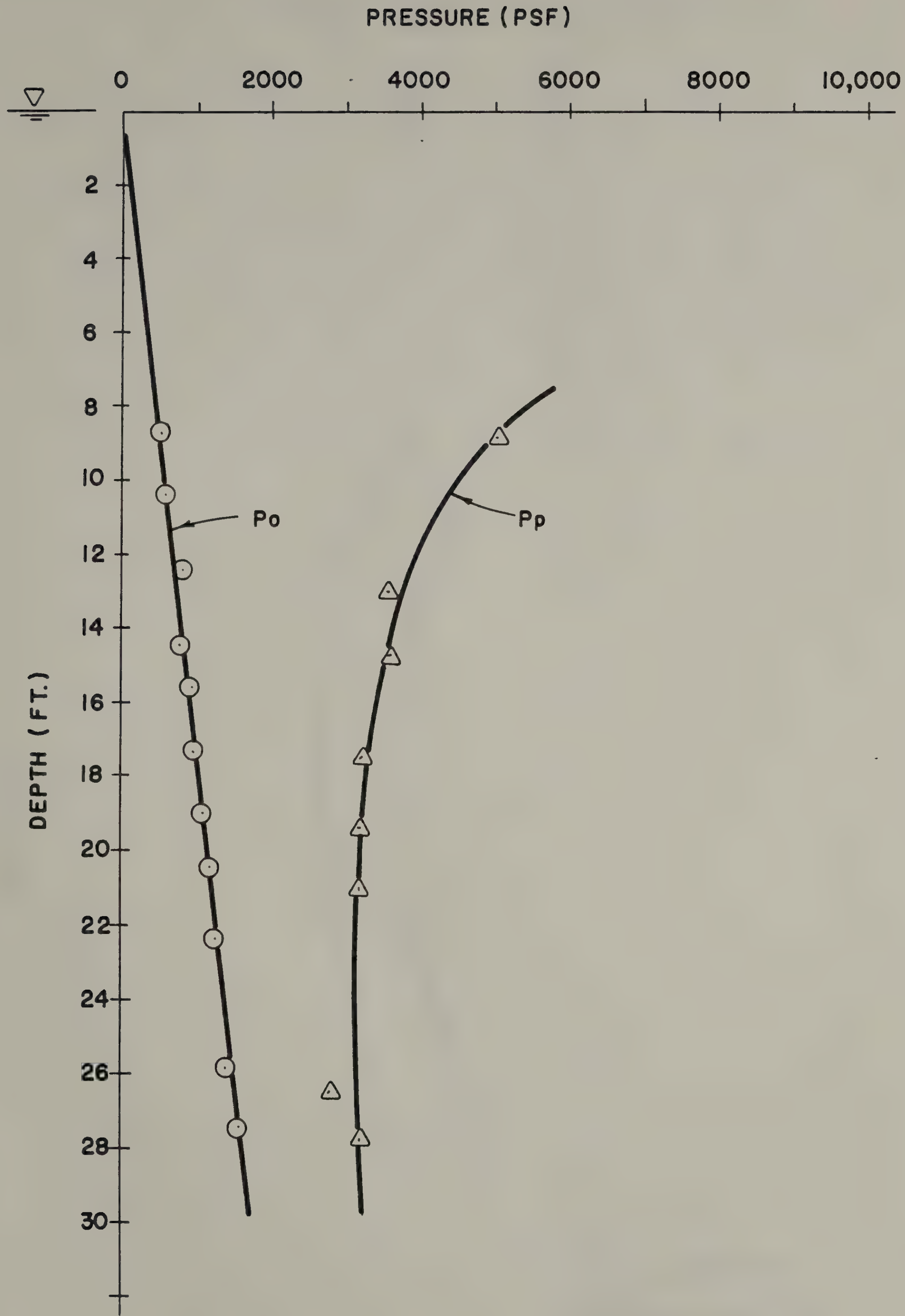


FIGURE 7: OVERBURDEN AND MAXIMUM PAST PRESSURE CURVES
(LOCKPORT CLAY)

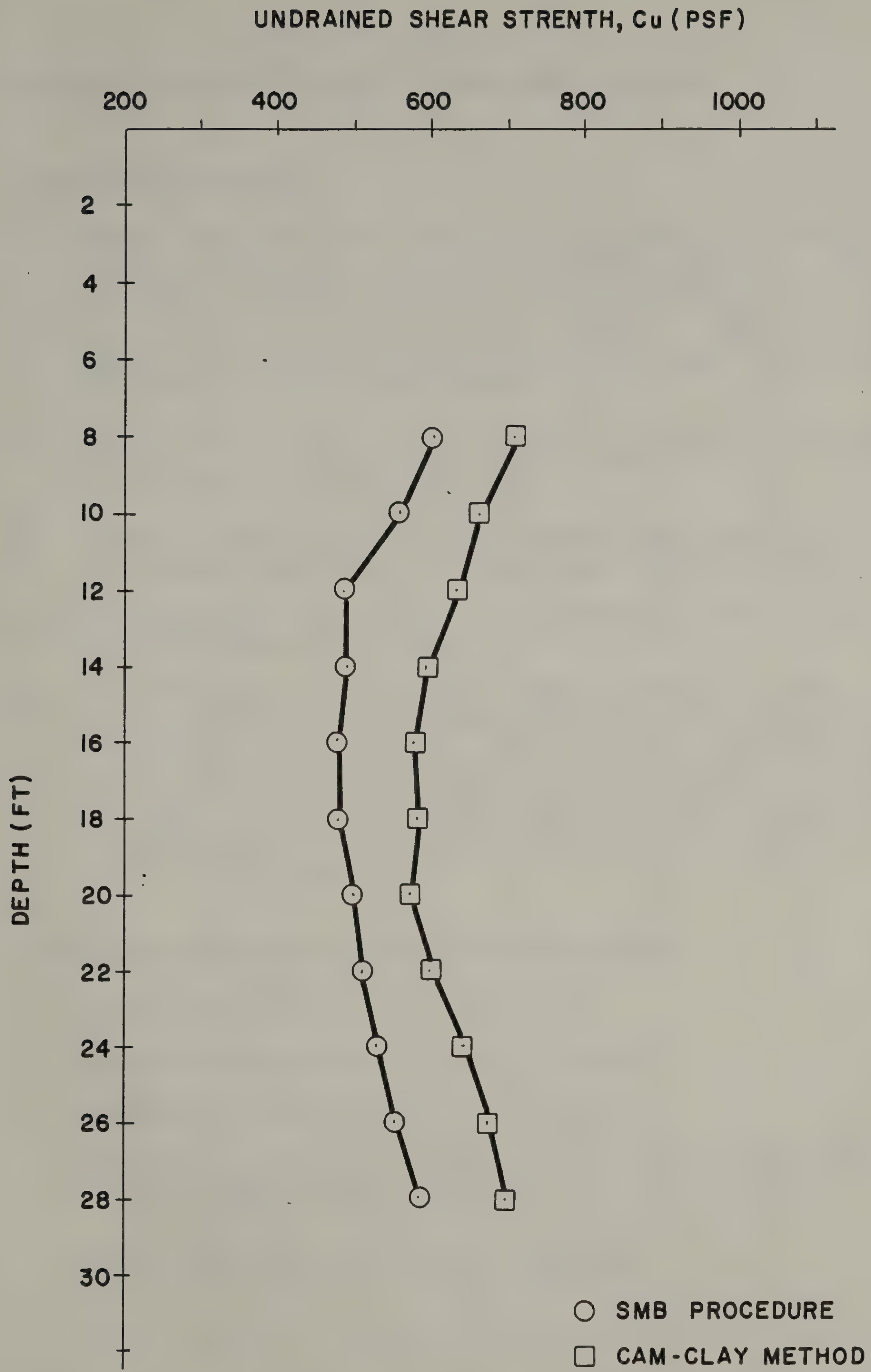


FIGURE 8: CAM-CLAY METHOD AND SMB PROCEDURE ;
UNDRAINED SHEAR STRENGTH vs DEPTH
(LOCKPORT CLAY)

PROCEDURE TO ESTIMATE OCR AND UNDRAINED SHEAR STRENGTH

USING TRIAXIAL TESTING:

1. Perform two CIU shear tests. In the first test, consolidate the specimen beyond the in-situ preconsolidation pressure. Generally, a consolidation pressure of 80 psi will be sufficient for clays in New York State. In the second test, consolidate the sample as in the first test, then rebound the sample to an OCR of 10. These two tests will measure the response of the specimen under normally consolidated and overconsolidated conditions, respectively.

2. Express the results of the CIU test in terms of normalized soil parameters, and plot the log of (C/P) vs. the log of OCR to establish a linear relationship (as in Appendix A).

3. Conduct a CIU test on a third specimen consolidated at its in-situ effective vertical overburden pressure and calculate (C/Po).

4. With the known (C/Po) from Step 3, find OCR from the log (C/P) vs. log OCR line established in Step 2.

5. To establish the variation of the undrained shear strength with depth, conduct a series of conventional consolidation tests to establish the distribution of Pp with depth. Then, with the distribution of OCR with depth known, the undrained shear strength of a deposit may be estimated from the log (C/P) vs. log OCR line established in Step 2 (graphical procedure) or by the procedure shown in the comparison of shear strengths obtained by Cam-Clay Method and SMB Procedure on Page 18 (analytical procedure).

USING CONSOLIDATION TESTING ONLY (NO TRIAXIAL TESTS):

1. Perform consolidation tests with rebounds.

2. Obtain parameters Cs and Cc from test results.

3. Determine Λ using; $\Lambda = 1 - (Cs/Cc)$ (Eq. 3)

4. Determine variation of undrained shear strength vs. depth using

$$\frac{C}{P} = \frac{3 \sin\phi'}{3 - \sin\phi'} \left(\frac{OCR}{e}\right)^{\Lambda} \quad \text{(Eq. 7)}$$

Perform calculation as in Table 1 (Page 18).

CONCLUSIONS

1. The majority of clays in New York State exhibit a "normalized behavior" which may be reasonably expressed by the Cam-Clay model. It has been found that Λ generally increases with increasing PI, although in some cases Λ decreases with increasing PI. Values of Λ obtained from triaxial testing range approximately from 0.4 to 0.9, with a median value of 0.75 for clays analyzed in this study.

2. It was originally anticipated that it would be possible to develop relationships for Λ vs. PI so that ultimately the equations in the Cam-Clay method could be solved without triaxial testing, provided OCR vs. depth for the deposit is given. Developing the Λ vs. PI relationships was only partly successful in that because of the moderate scatter in the data, good correlations could not be drawn, but rather only general trends. It is suspected that much of the scatter is attributable to the method by which the data was obtained.

3. The undrained shear strengths obtained by the Cam-Clay method are in good agreement with undrained shear strengths obtained from the SMB procedure. The Cam-Clay method is easy to use and only two CU tests are required to obtain the necessary parameters. The method provides a means by which the effects of overconsolidation on the undrained shear strength can be directly considered in the analysis.

4. At this point, it appears that it may be easier, more convenient, and more economical to obtain Λ from consolidation testing than from triaxial testing. However, further study is required to confirm this assumption.

RECOMMENDATIONS FOR FURTHER STUDY

Theoretically, it should be more accurate to obtain Λ from Equation (3), which describes the original relationship, than to obtain Λ from equations derived from Equation (1). However, because of the problems associated with obtaining the parameters in Equation (3), this may not necessarily be true. In order to investigate this situation, it is recommended that an additional study be conducted that addresses the following items:

1) Compare Λ derived from the procedure presented in this paper with Λ derived from Equation (3).

2) Obtain the parameters Cs and Cc from the conventional consolidation tests using the TACT system because of its high degree of automation and time savings.

3) Develop a procedure to find a reliable point on a rebounded e-log P curve through which a line can be drawn to approximate Cs.

4) Determine if Cs can be approximated by a constant value without a loss in the degree of accuracy.

5) Determine if Cs is significantly influenced by variations in the location of the rebound curve along the virgin compression curve.

6) Investigate the sensitivity of the parameters to sample disturbance.

7) Compare the results of various procedures to field records.

APPENDIX A

(C/P) VS. OCR TO DETERMINE Λ FOR REGION 1 CLAYS, LOCKPORT CLAY AND WESTWAY CLAY

NOTE: All tests are CIU Triaxial Tests unless otherwise indicated.

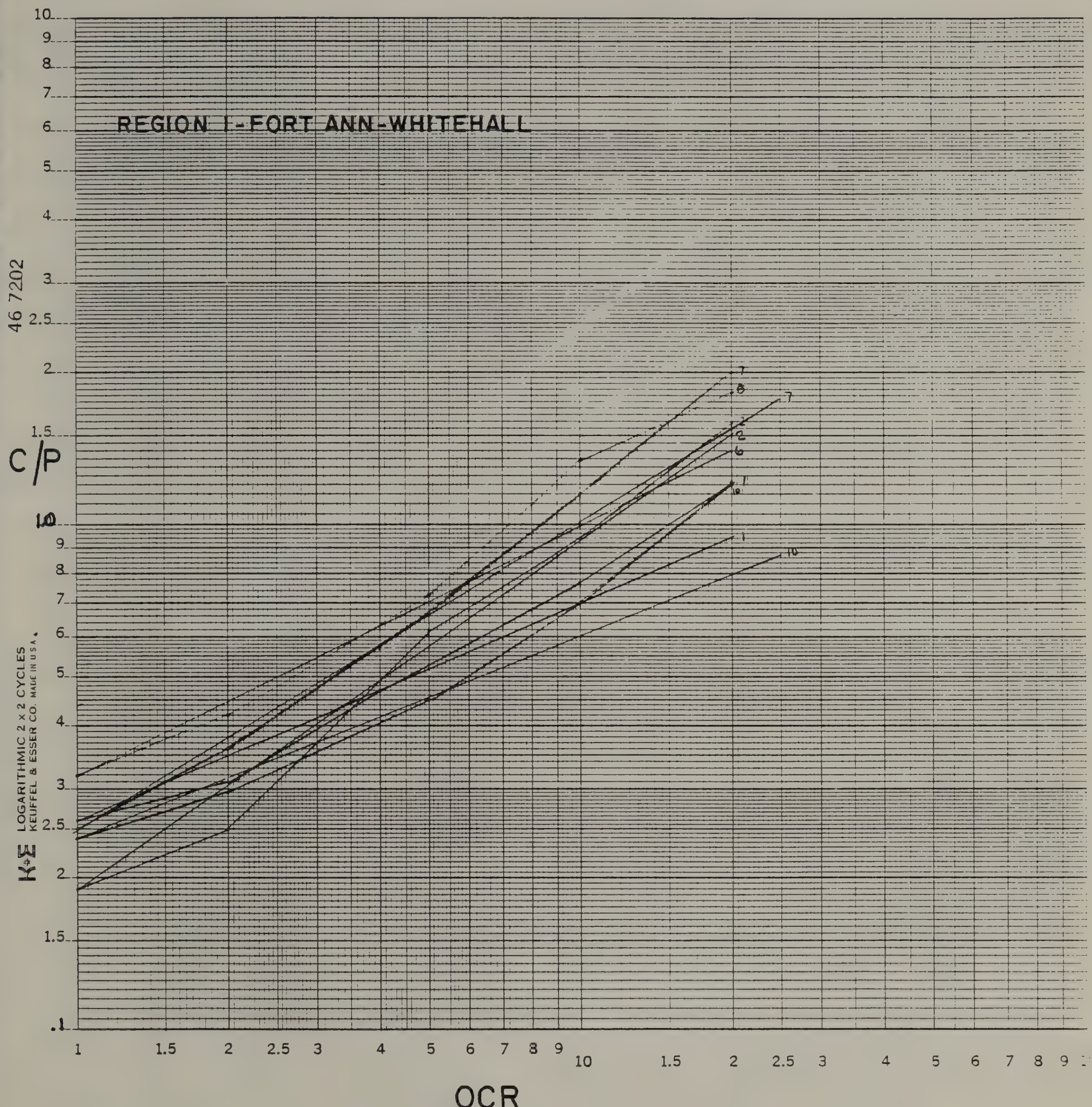


FIGURE A1: C/P vs OCR TO DETERMINE Λ

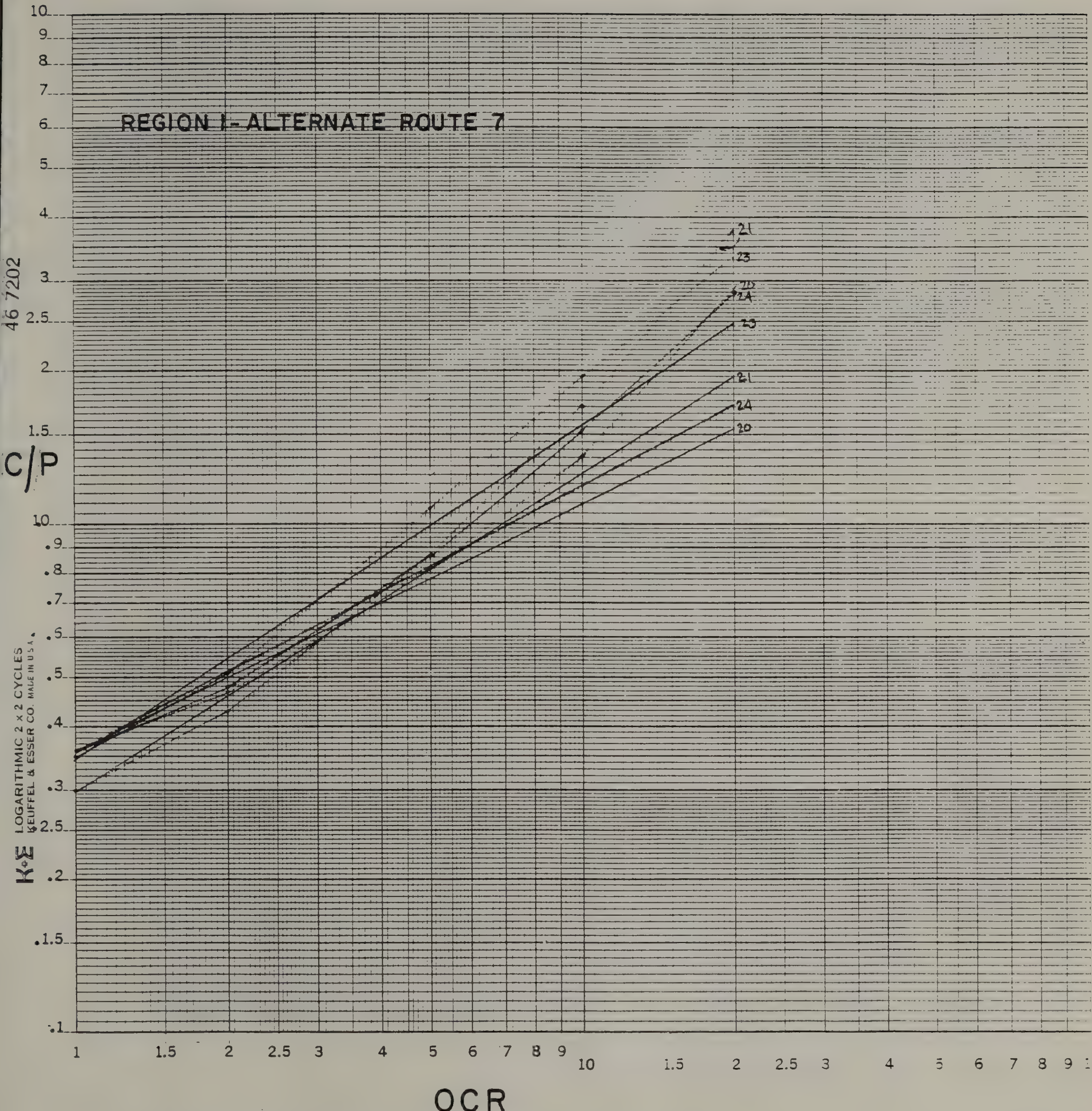


FIGURE A2: C/P vs OCR TO DETERMINE Λ

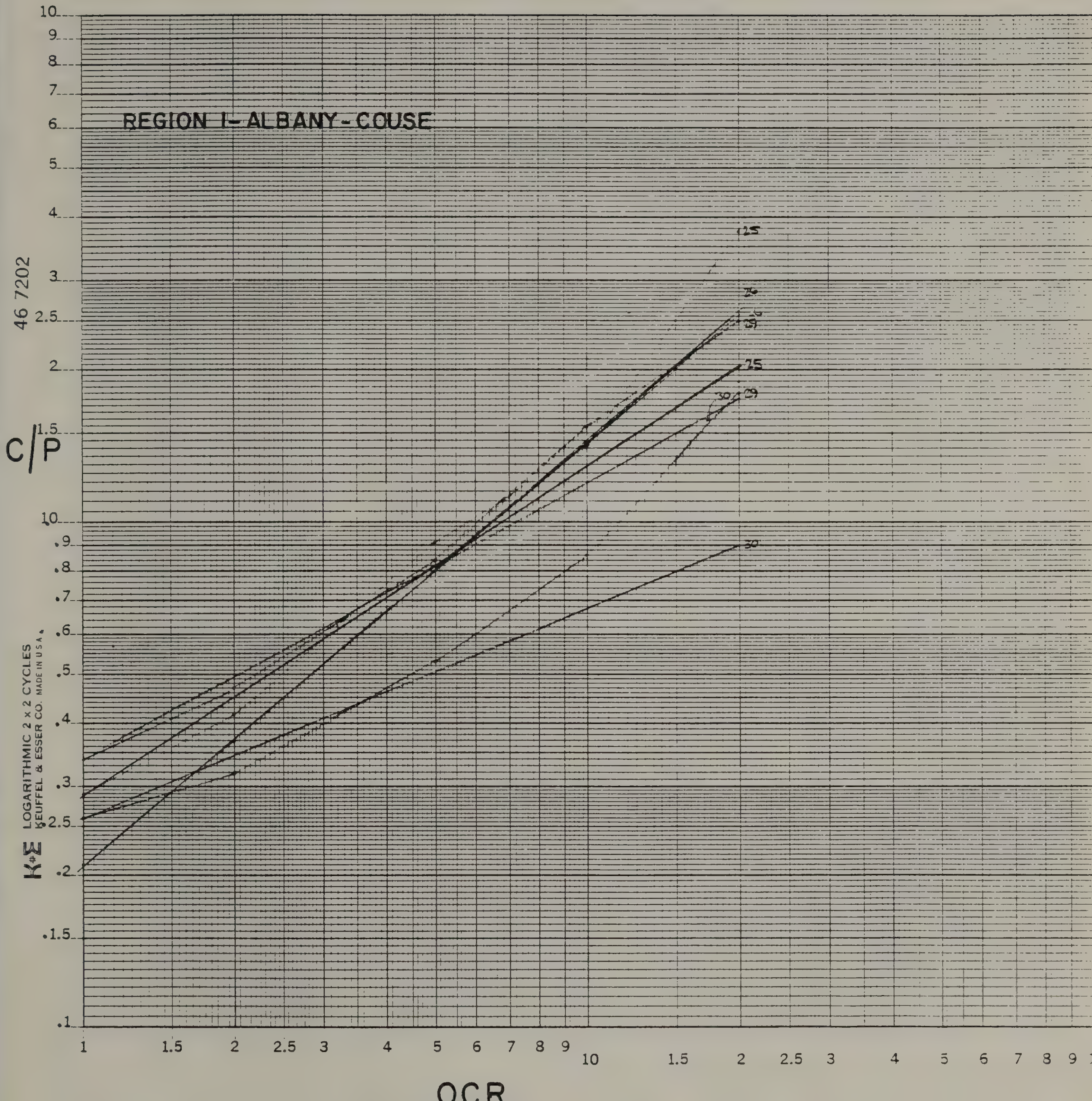


FIGURE A3: C/P vs OCR TO DETERMINE Λ

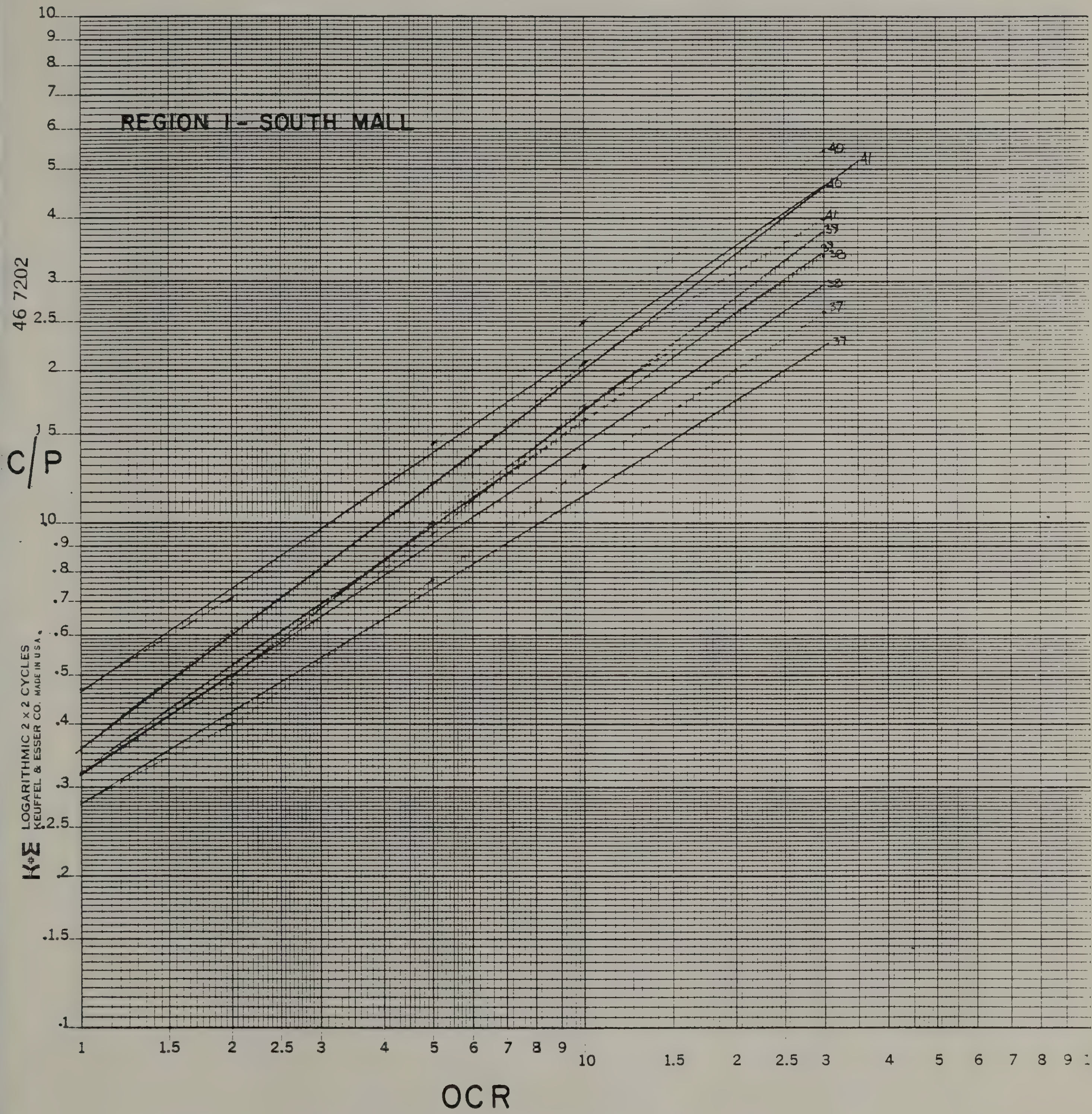


FIGURE A4: C/P vs OCR TO DETERMINE Λ

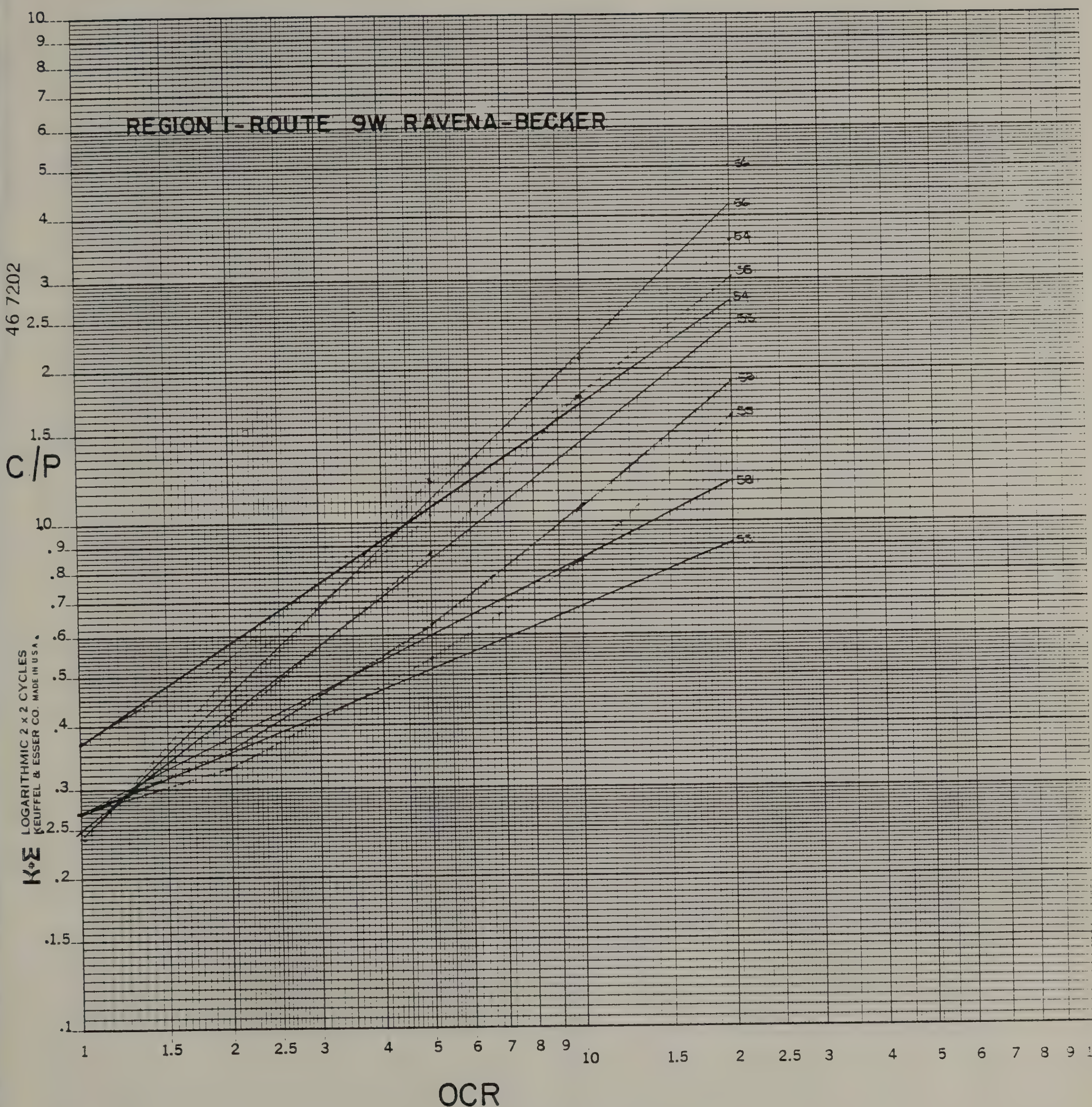


FIGURE A5: C/P vs OCR TO DETERMINE Λ

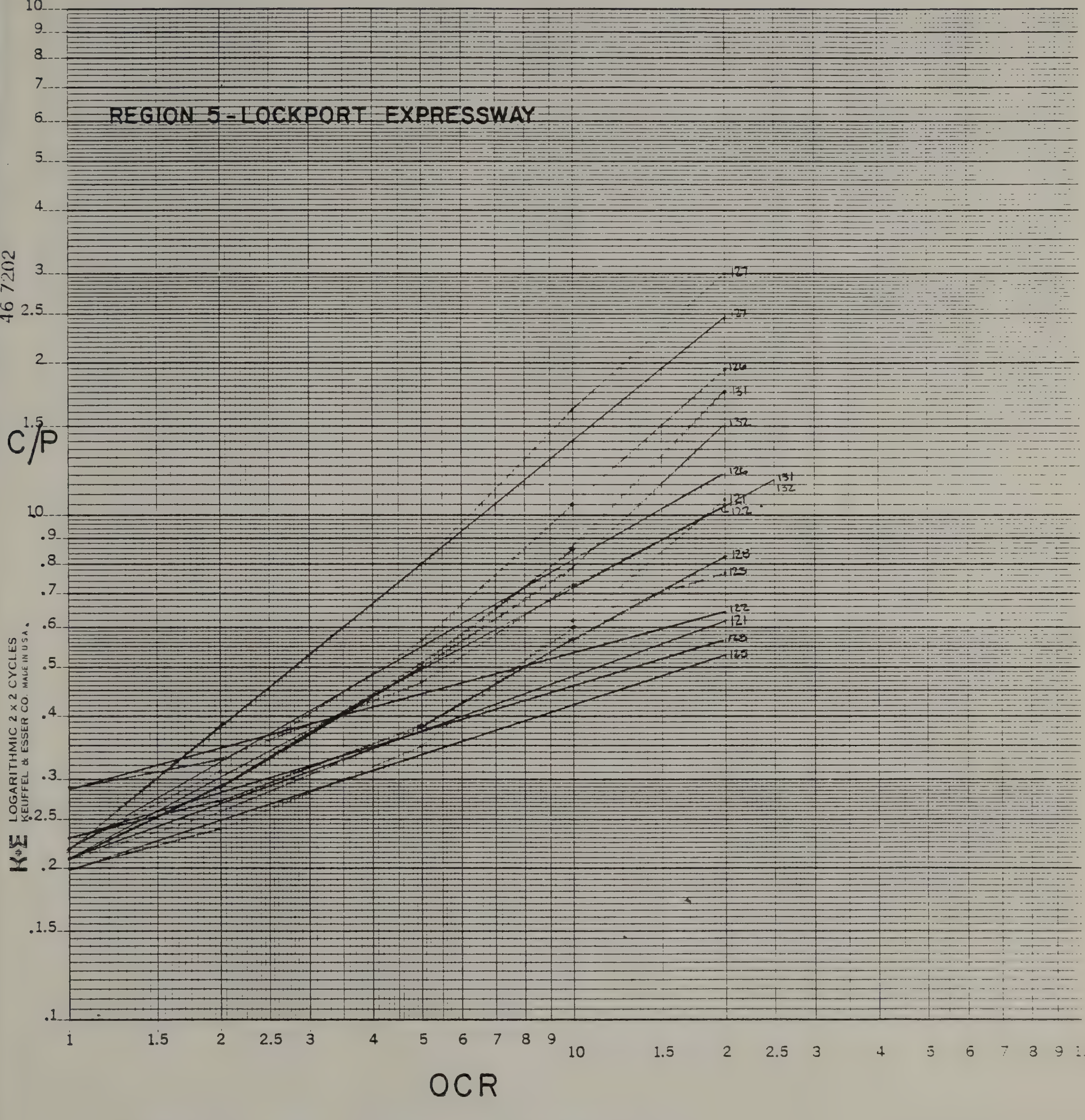


FIGURE A6: C/P vs OCR TO DETERMINE Λ

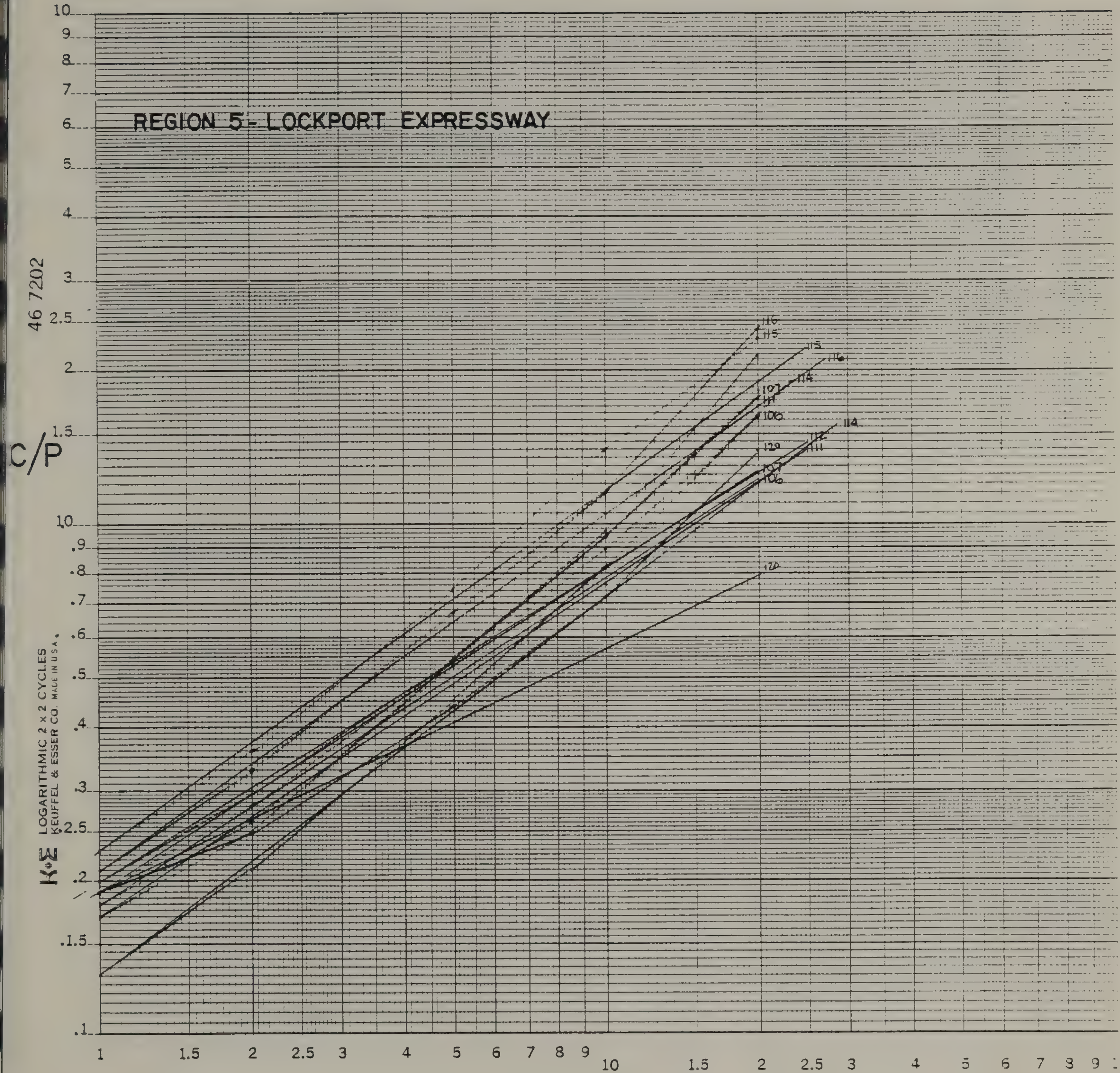


FIGURE A7: C/P vs OCR TO DETERMINE Λ

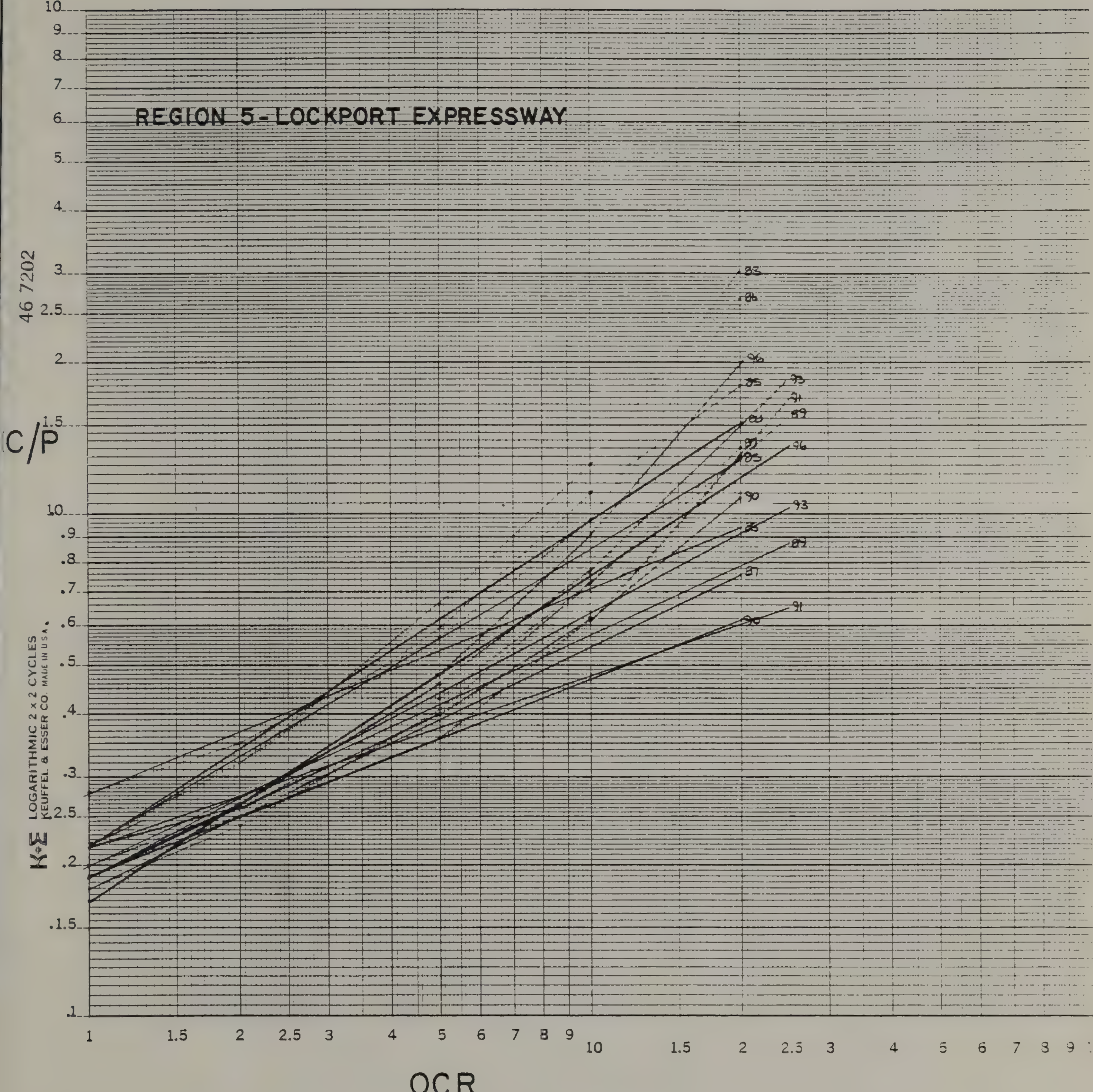


FIGURE A8: C/P vs OCR TO DETERMINE Λ

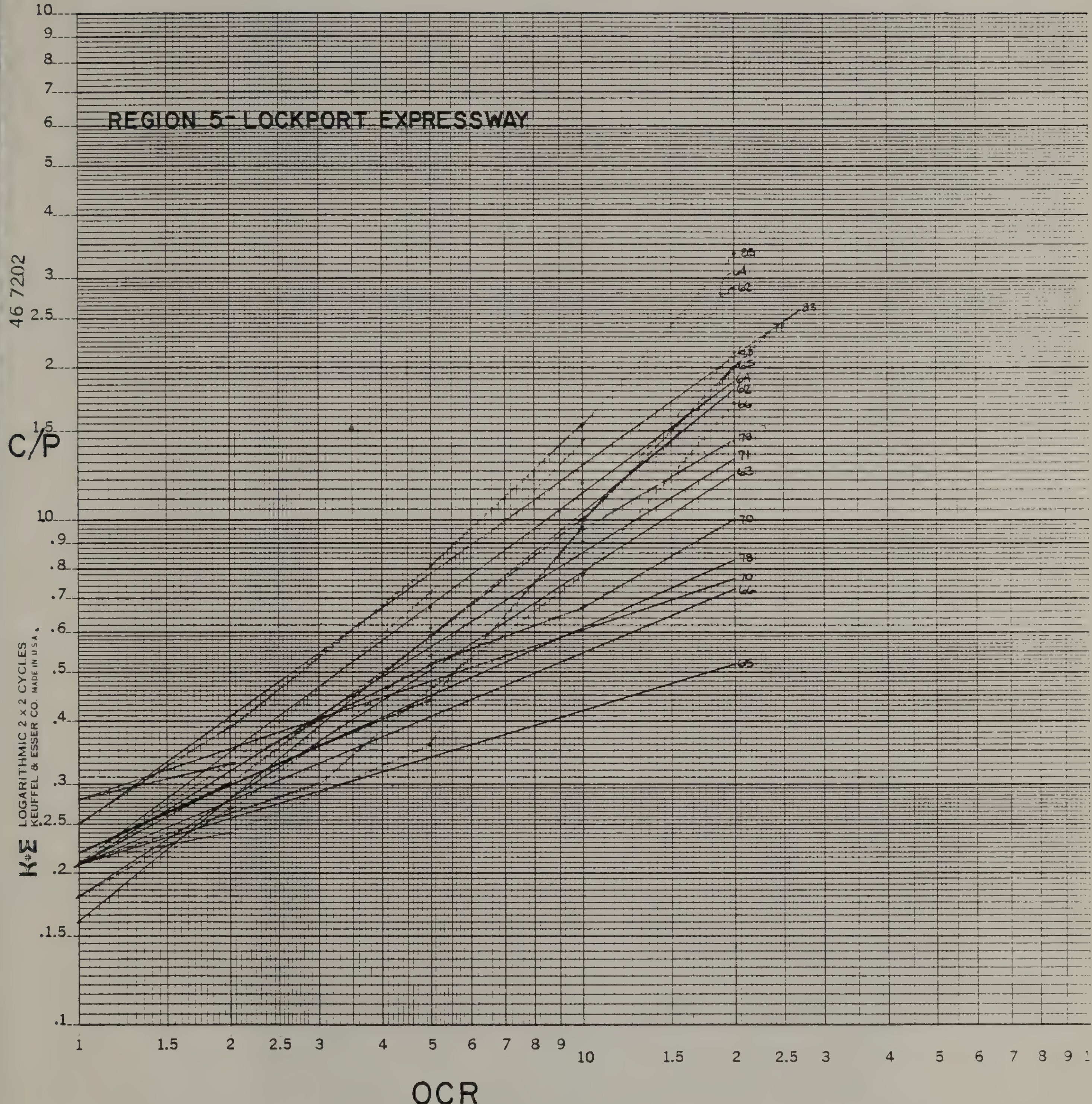
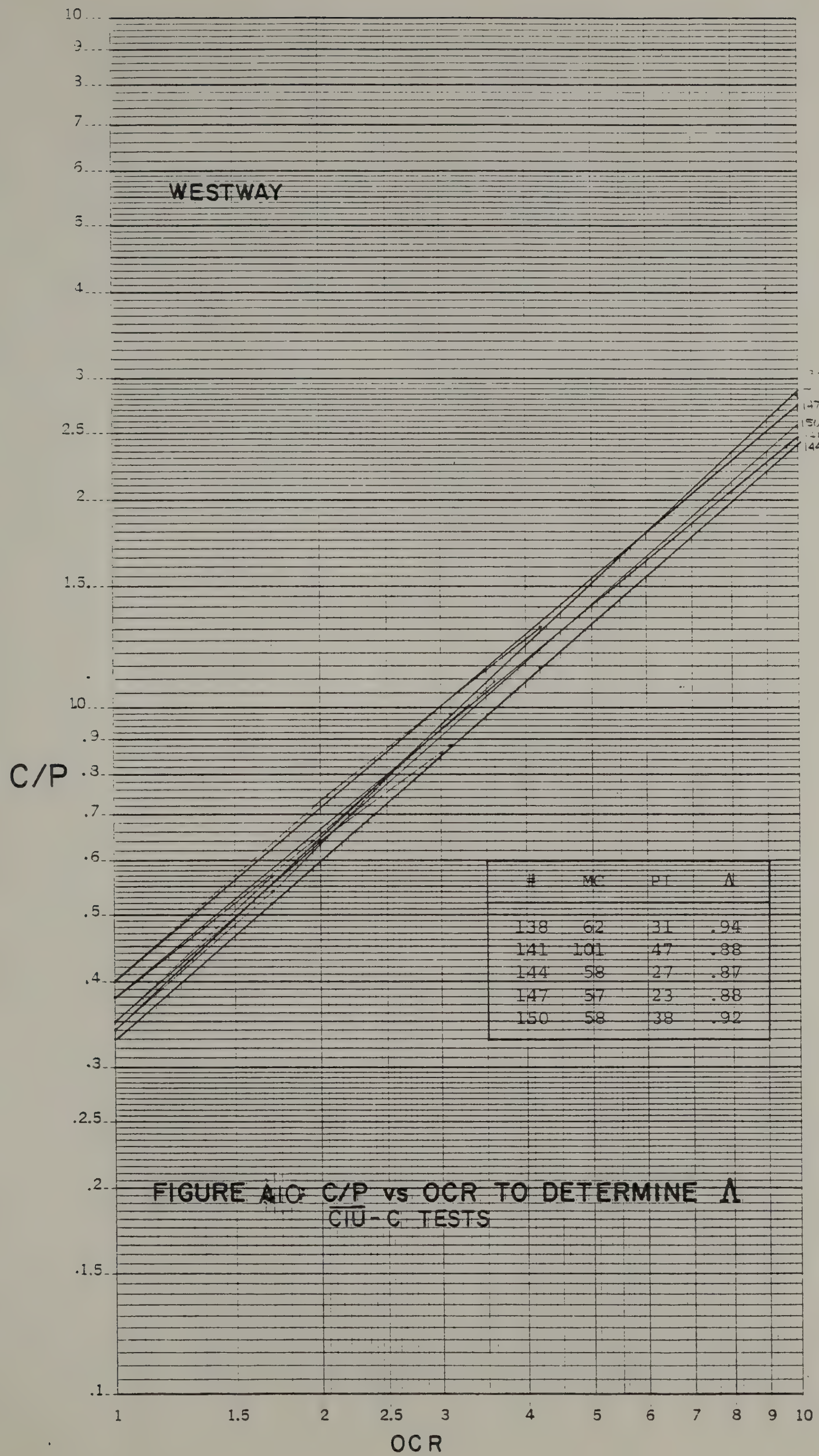


FIGURE A9: C/P vs OCR TO DETERMINE Λ



#	MC	PI	Λ
138	62	31	.94
141	101	47	.88
144	58	27	.87
147	57	23	.88
150	58	38	.92

FIGURE A10: C/P vs OCR TO DETERMINE Λ
$\overline{CIU}$-C TESTS

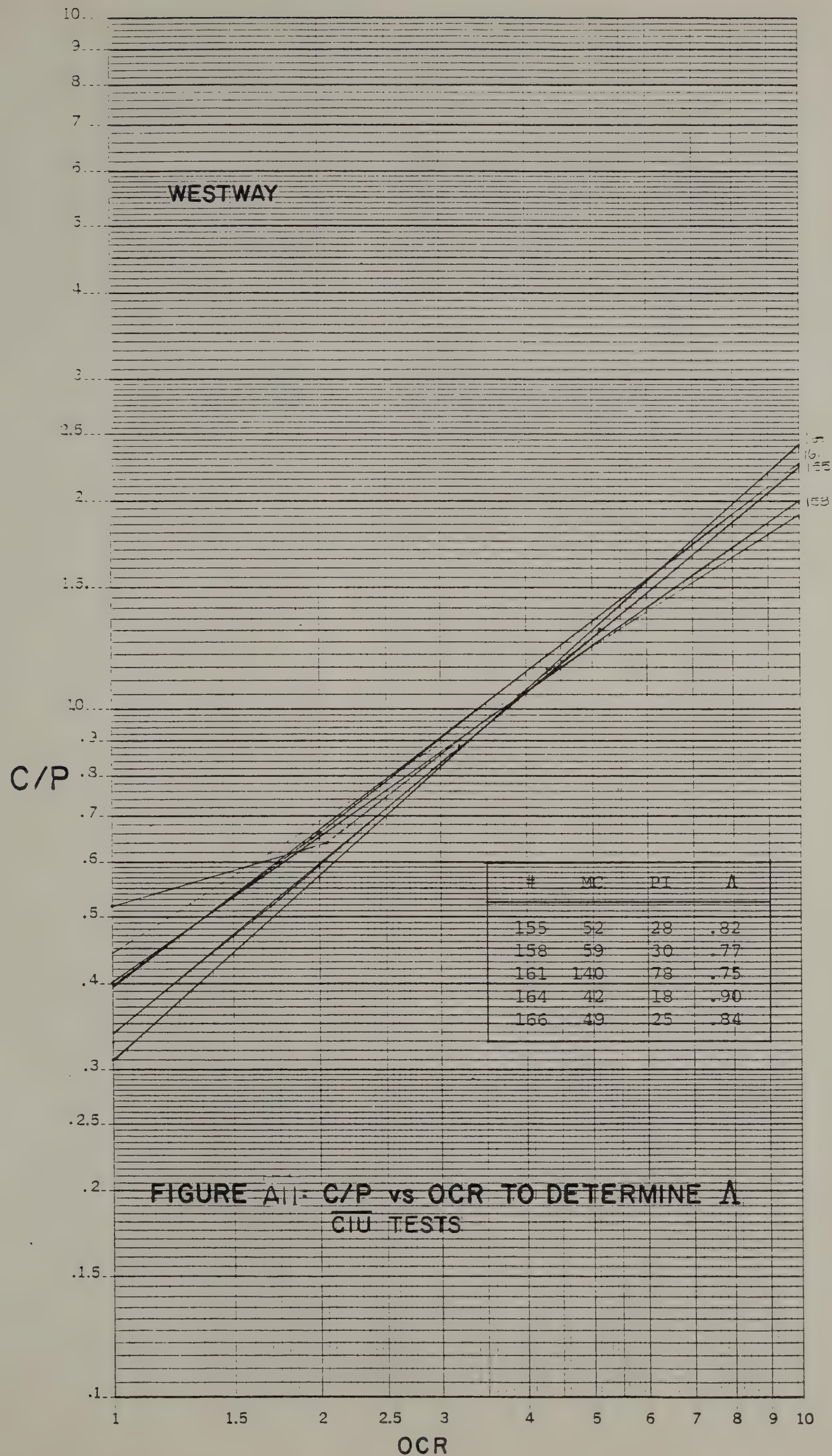


FIGURE A11: C/P vs OCR TO DETERMINE Λ
$\overline{\text{CIU}}$ TESTS

#	MC	PI	Λ
155	52	28	.82
158	59	30	.77
161	140	78	.75
164	42	18	.90
166	49	25	.84

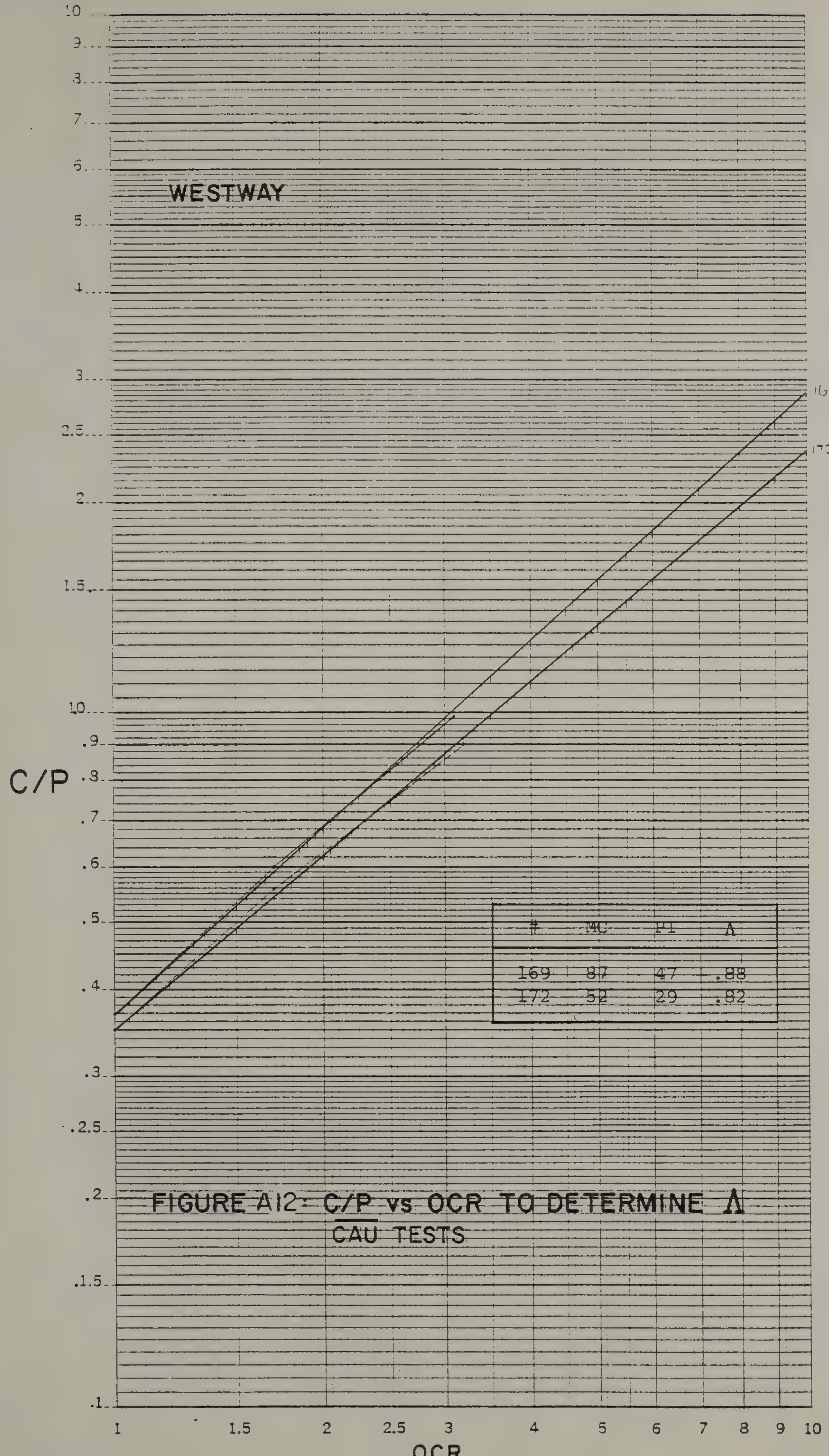


FIGURE A12: C/P vs OCR TO DETERMINE Λ
CAU TESTS

APPENDIX B

REGRESSION ANALYSES FOR SELECTED PROJECTS

NOTE: Regression analyses and plots are derived from the STATGRAPHICS Statistical Graphics System.
r = Sample Correlation Coefficient
n = Number of Samples

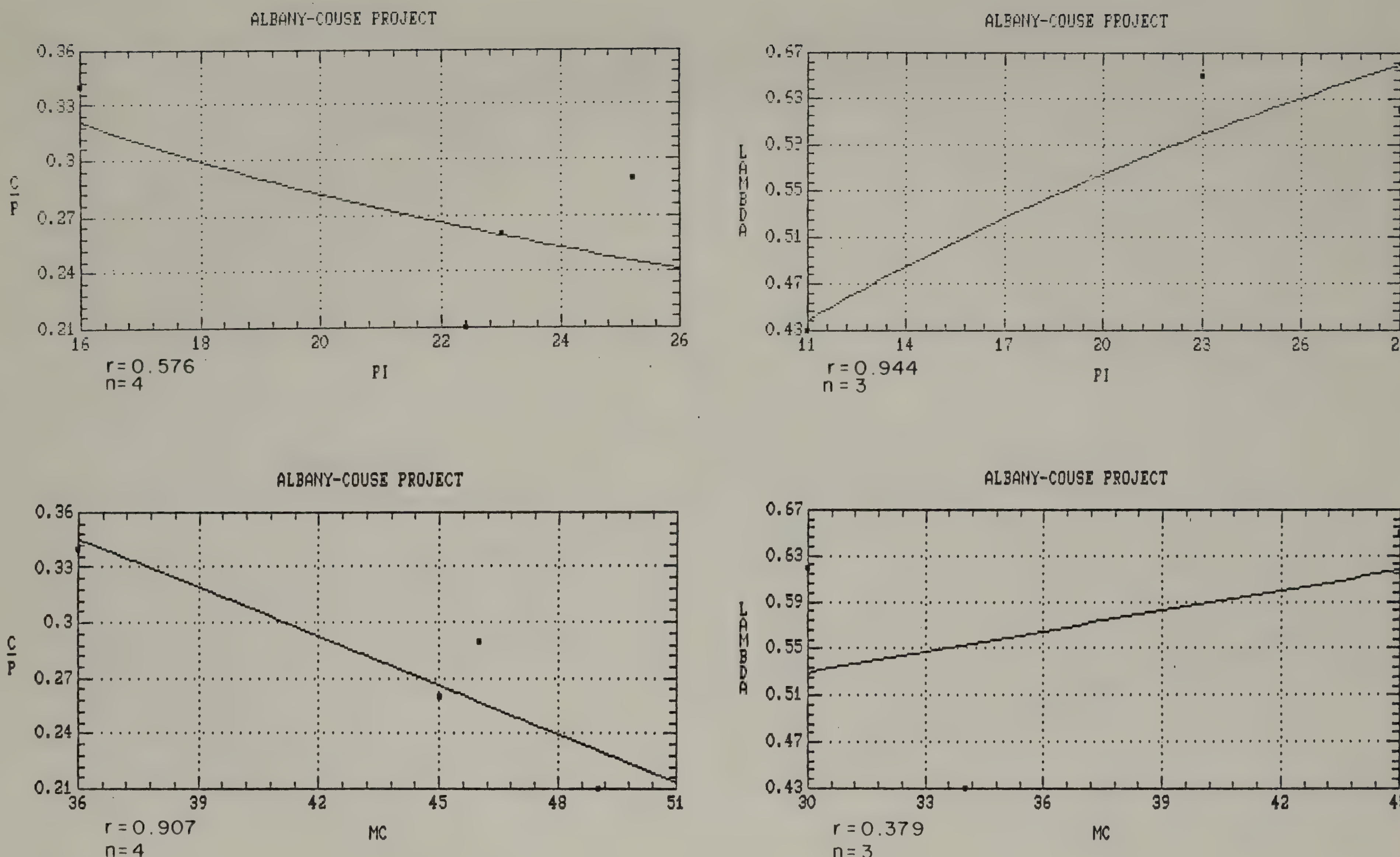

ALBANY-COUSE PROJECT
C/P
PI
r = 0.576
n = 4
ALBANY-COUSE PROJECT
LAMBDA
PI
r = 0.944
n = 3
ALBANY-COUSE PROJECT
C/P
MC
r = 0.907
n = 4
ALBANY-COUSE PROJECT
LAMBDA
MC
r = 0.379
n = 3

FIGURE BI

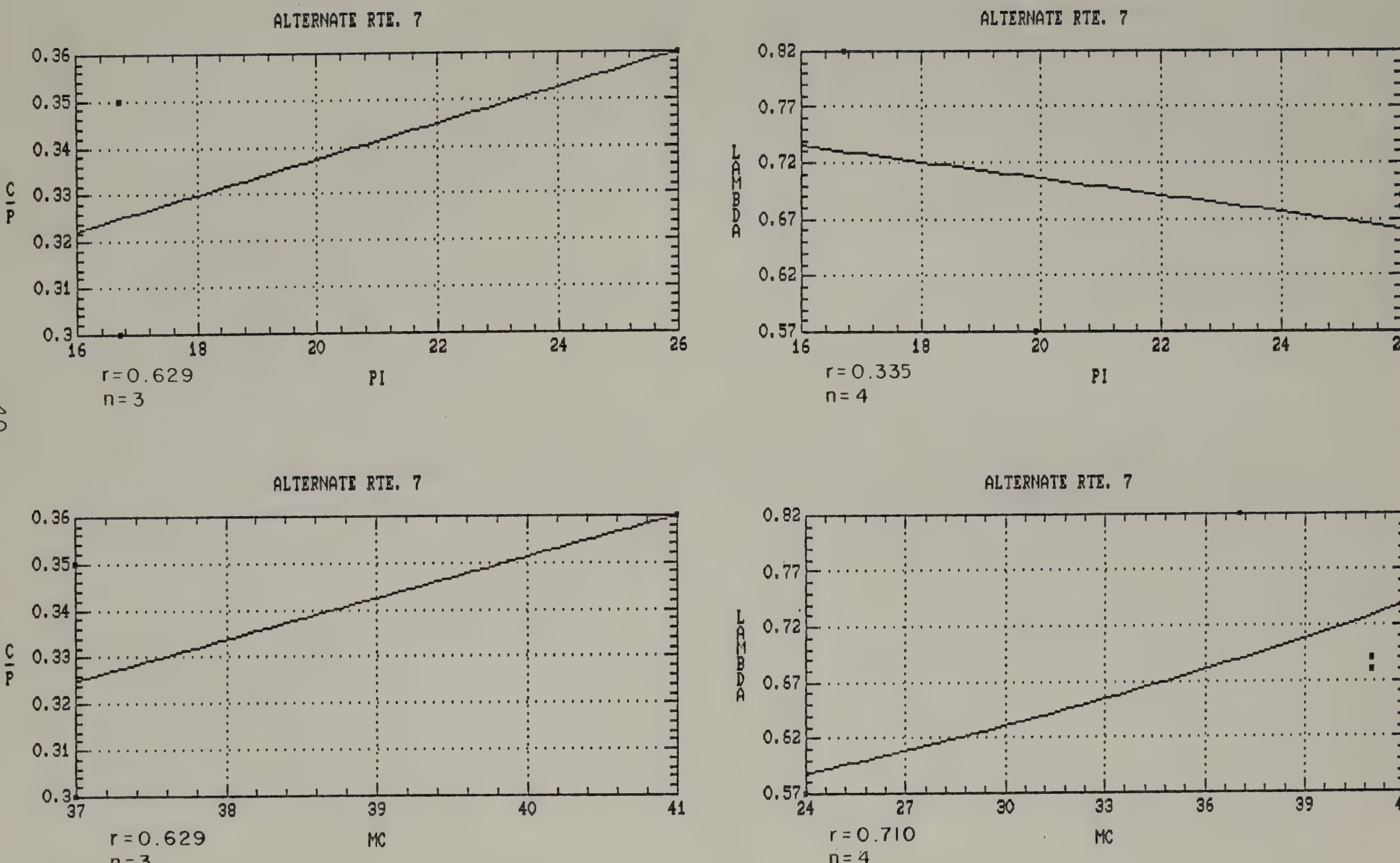

ALTERNATE RTE. 7
C/P
0.36
0.35
0.34
0.33
0.32
0.31
0.3
16
18
20
22
24
26
PI
r=0.629
n=3
ALTERNATE RTE. 7
LAMBDA
0.82
0.77
0.72
0.67
0.62
0.57
16
18
20
22
24
26
PI
r=0.335
n=4
ALTERNATE RTE. 7
C/P
0.36
0.35
0.34
0.33
0.32
0.31
0.3
37
38
39
40
41
MC
r=0.629
n=3
ALTERNATE RTE. 7
LAMBDA
0.82
0.77
0.72
0.67
0.62
0.57
24
27
30
33
36
39
42
MC
r=0.710
n=4

FIGURE B2

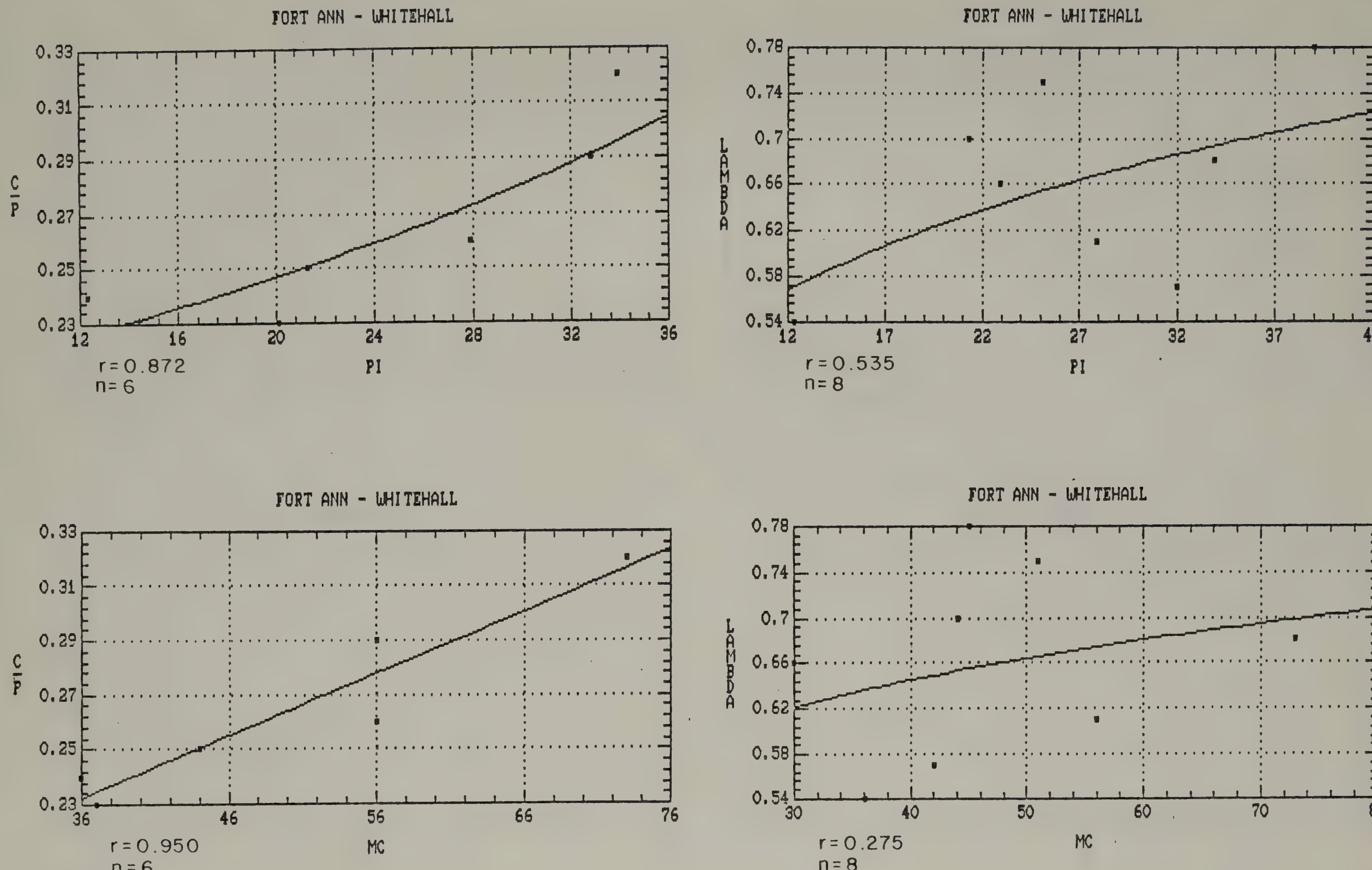

FORT ANN - WHITEHALL
C/P
PI
r = 0.872
n = 6
FORT ANN - WHITEHALL
LAMBDA
PI
r = 0.535
n = 8
FORT ANN - WHITEHALL
C/P
MC
r = 0.950
n = 6
FORT ANN - WHITEHALL
LAMBDA
MC
r = 0.275
n = 8

FIGURE B3

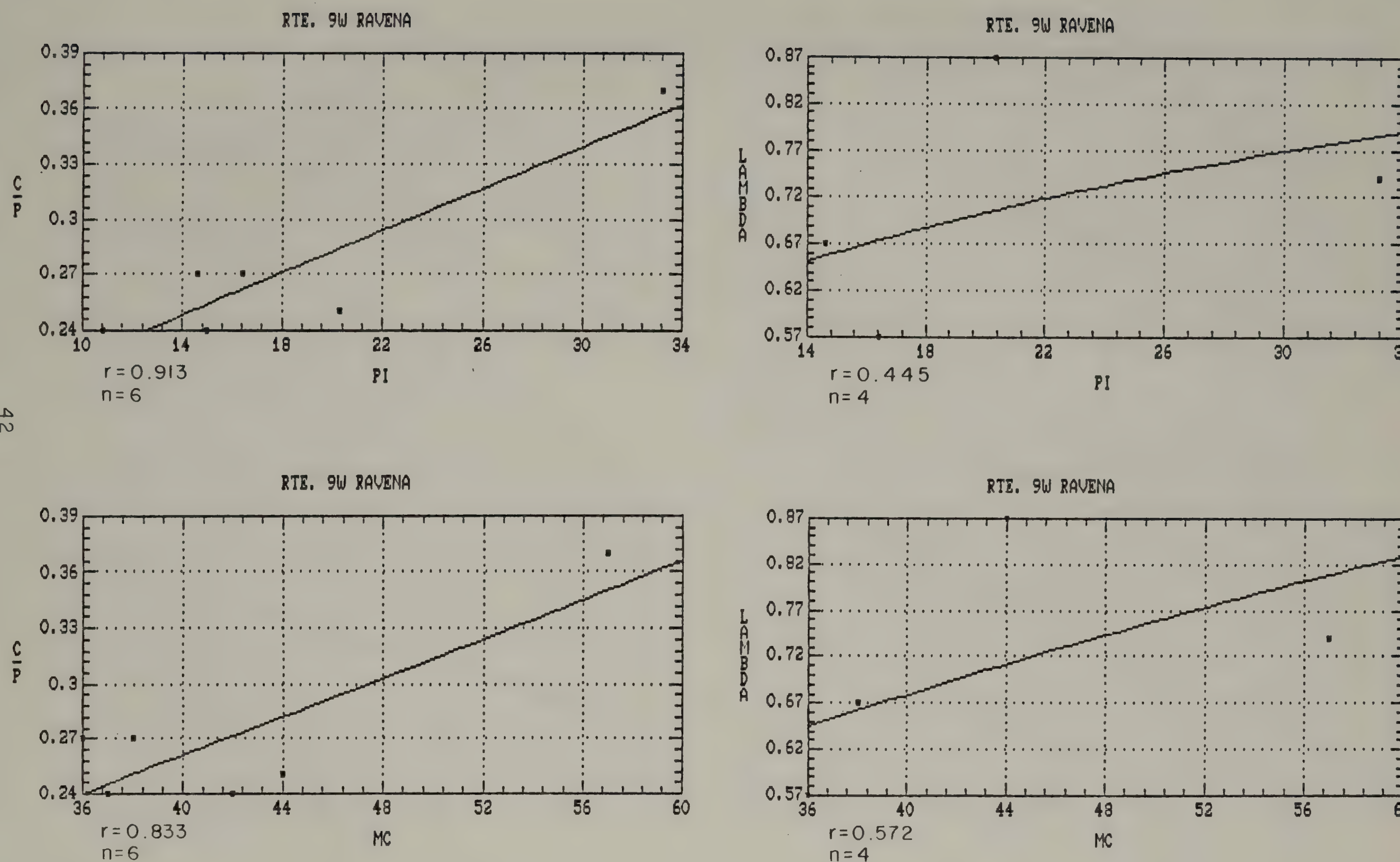

RTE. 9W RAVENA
C/P
PI
r=0.913
n=6
RTE. 9W RAVENA
LAMBDA
PI
r=0.445
n=4
RTE. 9W RAVENA
C/P
MC
r=0.833
n=6
RTE. 9W RAVENA
LAMBDA
MC
r=0.572
n=4

FIGURE B4

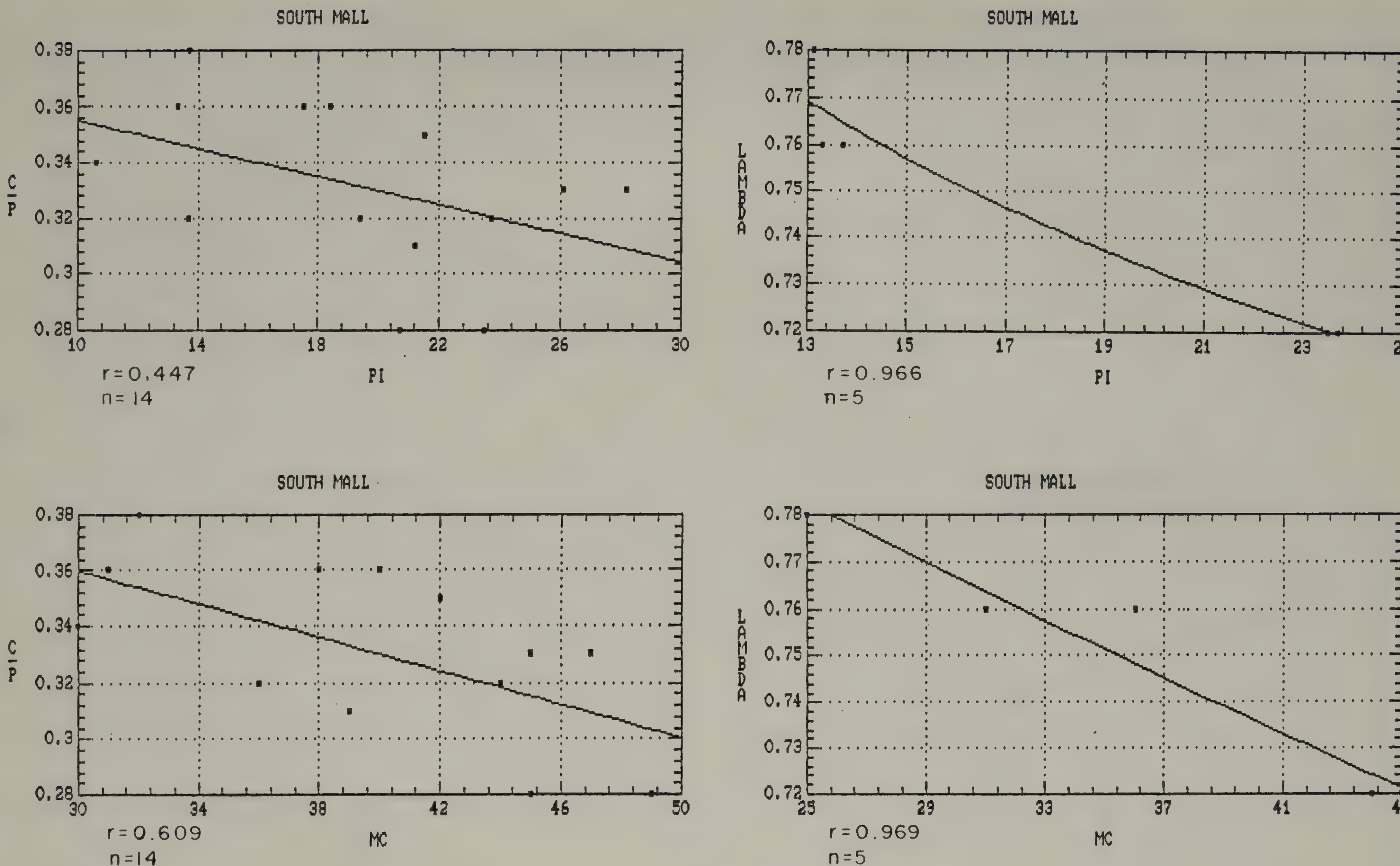

SOUTH MALL
C/P
PI
r=0.447
n=14
SOUTH MALL
LAMBDA
PI
r=0.966
n=5
SOUTH MALL
C/P
MC
r=0.609
n=14
SOUTH MALL
LAMBDA
MC
r=0.969
n=5

FIGURE B5

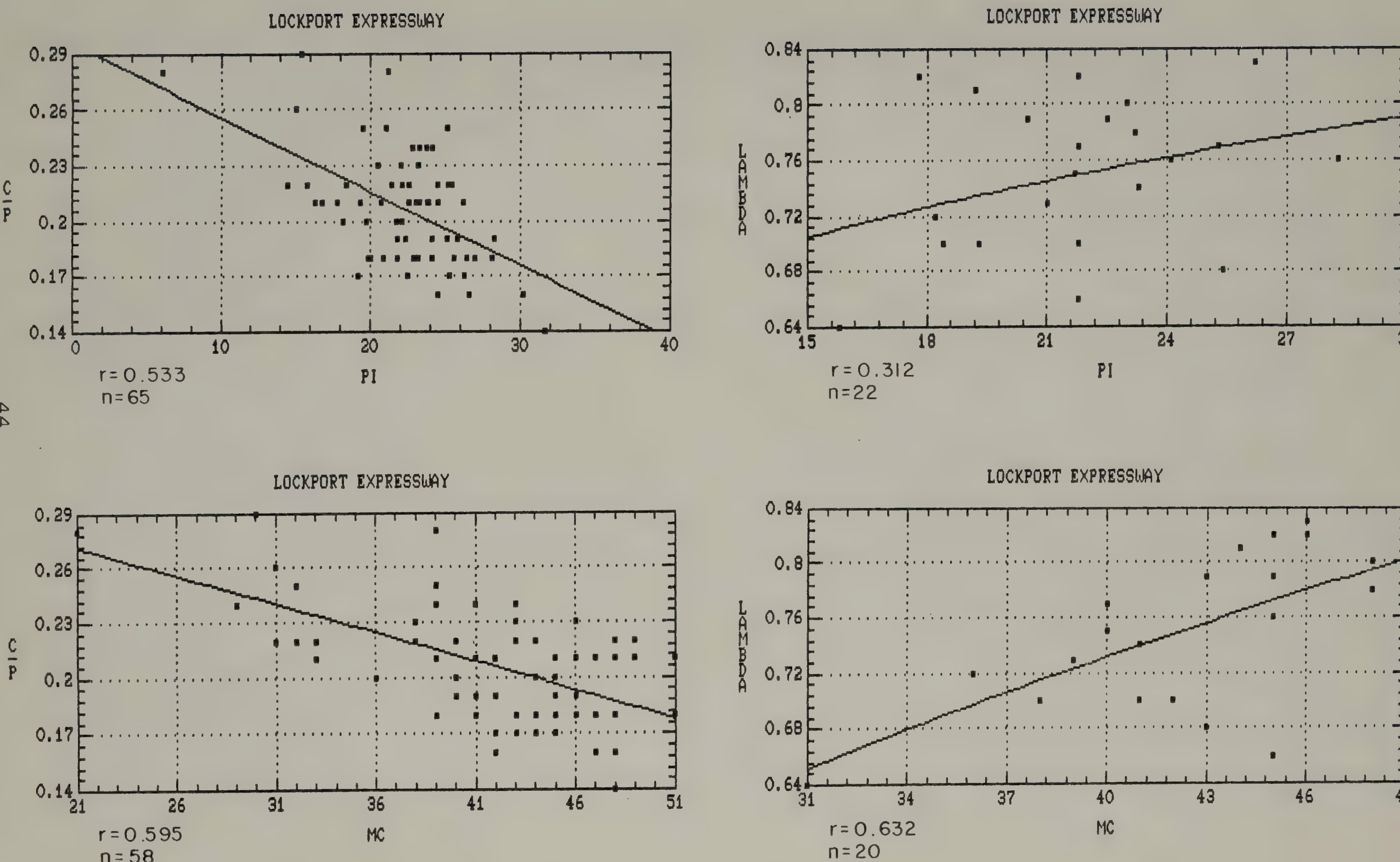

LOCKPORT EXPRESSWAY
C/P
PI
r=0.533
n=65
LOCKPORT EXPRESSWAY
LAMBDA
PI
r=0.312
n=22
LOCKPORT EXPRESSWAY
C/P
MC
r=0.595
n=58
LOCKPORT EXPRESSWAY
LAMBDA
MC
r=0.632
n=20

FIGURE B6

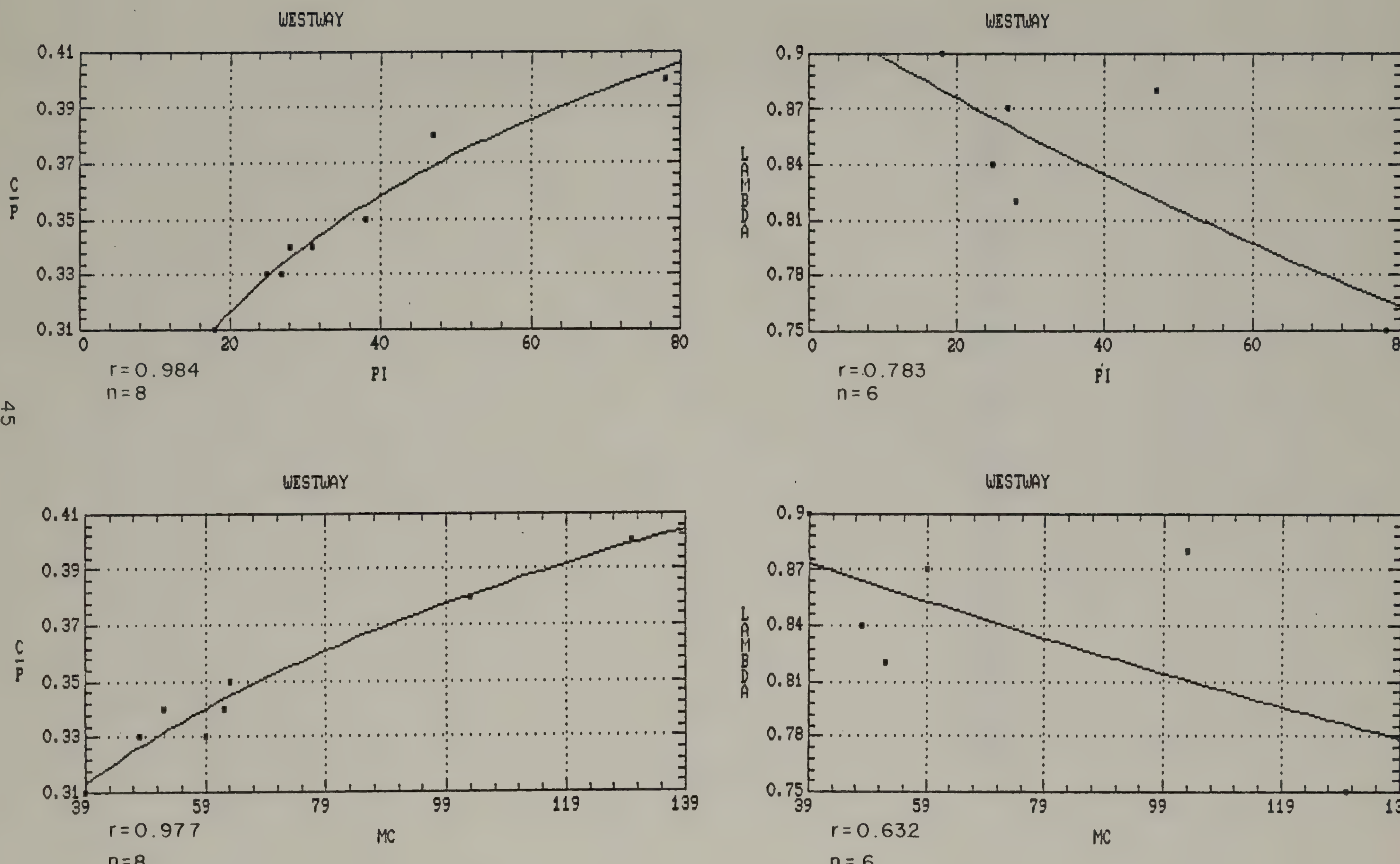

WESTWAY
C/P
0.41
0.39
0.37
0.35
0.33
0.31
0
20
40
60
80
PI
r=0.984
n=8
WESTWAY
LAMBDA
0.9
0.87
0.84
0.81
0.78
0.75
0
20
40
60
80
PI
r=0.783
n=6
WESTWAY
C/P
0.41
0.39
0.37
0.35
0.33
0.31
39
59
79
99
119
139
MC
r=0.977
n=8
WESTWAY
LAMBDA
0.9
0.87
0.84
0.81
0.78
0.75
39
59
79
99
119
139
MC
r=0.632
n=6

FIGURE B7

APPENDIX C

PLOT SHOWING INFLUENCE OF PLASTICITY INDEX ON EFFECTIVE ANGLE OF INTERNAL FRICTION USING BOTH DRAINED AND UNDRAINED TESTS

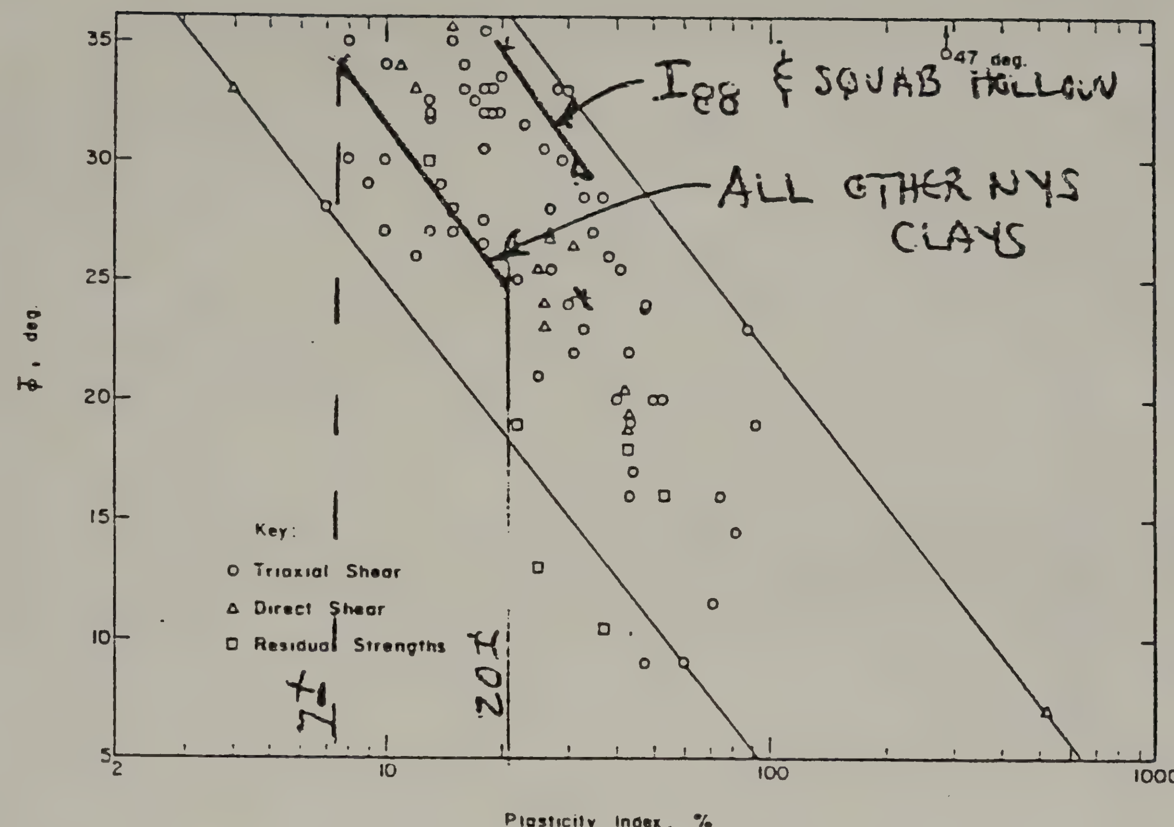


FIG. CI —Influence of Plasticity Index on Effective Angle of Internal Friction as Determined Using Both Drained and Undrained Tests (after Olson, 1976)

APPENDIX D

SUMMARY OF DATA COLLECTED

NOTE: All tests are CIU Triaxial Tests unless otherwise indicated.

The data for this study was obtained by the following procedure:

1. Estimate Pp by the procedure shown on Page 7, Figure 2.

2. Divide Pp by 2, 5, 10 and 20, and mark the intercept of these pressures on the Failure Envelope. These values will provide points at adequate intervals through which a line can be drawn for the C/P vs. OCR relationship.

3. Extend lines from the origin through each point on the Failure Envelope.

4. The slope of each line is the (C/P) for each of the corresponding OCR's.

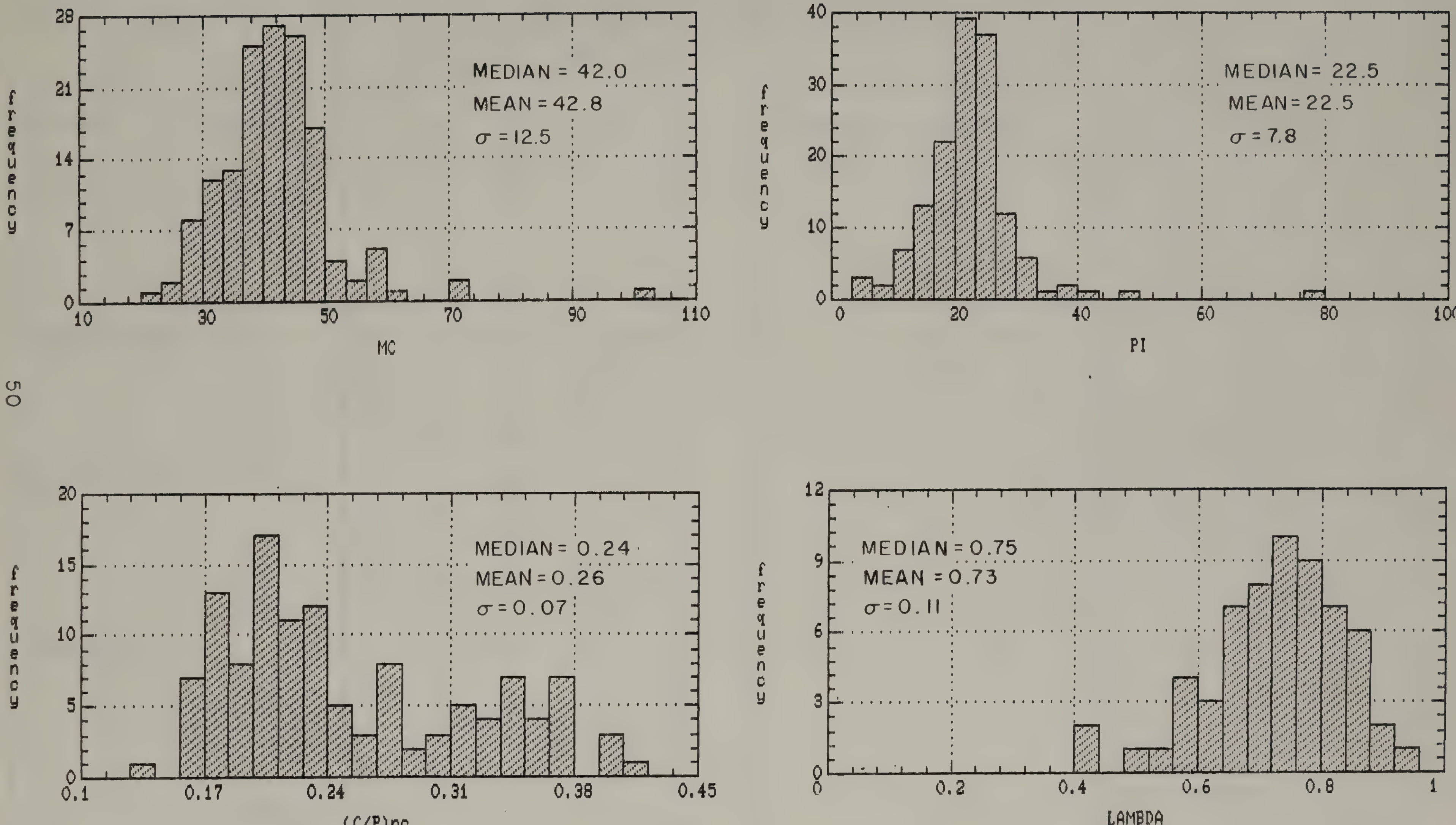


FIGURE DI: FREQUENCY HISTOGRAMS FOR MOISTURE CONTENT (MC), PLASTICITY INDEX (PI), (C/P)nc, AND LAMBDA (Λ) FOR ALL DATA COLLECTED

TABLE DI: SUMMARY OF SAMPLE PROPERTIES AND Λ READ FROM PLOTS

PROJECT	I.D.	MC	PI	Λ	
				MAX	MIN
FORT ANN - WHITEHALL	1	56	27.9	.61	.43
	2	51	25.1	.75	.69
	3	42	32.0	.57	.24
	4	45	39.0	.78	.58
	6	73	33.9	.68	.50
	7	44	21.3	.70	.60
	9	30	22.9	.66	.50
	10	36	12.3	.54	.40
ALTERNATE ROUTE 7	20	41	26.0	.68	.48
	21	37	16.7	.84	.62
	22	24	19.9	.57	.39
	23	37	16.7	.82	.65
	24	41	26.0	.69	.52
ALBANY - COUSE	25	46	25.2	.86	.65
	26	49	22.4	.84	.84
	27	34	11.0	.43	.23
	28	30	29.0	.62	.54
	29	36	16.0	.66	.55
	30	45	23.0	.65	.42
SOUTH MALL	37	45	23.5	.72	.61
	38	36	19.4	.79	.66
	39	36	13.7	.76	.70
	40	25	13.1	.78	.67
	41	31	13.3	.76	.74
	42	44	23.7	.72	.61
ROUTE 9W RAVENA - BECKER	54	57	33.2	.74	.66
	55	36	16.4	.57	.40
	56	37	10.8	.98	.95
	57	44	20.3	.87	.76
	58	38	14.6	.67	.49

PROJECT	I.D.	MC	PI	Λ	
				MAX	MIN
LOCKPORT EXPRESSWAY	62	48	26.6	.92	.81
	63	46	21.8	.82	.64
	64	47	22.6	.92	.73
	65	46	26.2	.83	.30
	66	42	19.3	.70	.42
	70	40	20.8	.44	.34
	71	41	23.3	.74	.61
	78	43	25.4	.68	.45
	83	39	21.0	.73	.71
	85	40	25.3	.77	.59
	86	37	25.1	.88	.40
	87	45	24.1	.76	.48
	88	38	18.4	.70	.64
	89	36	18.2	.72	.46
	90	40	21.8	.77	.39
	91	31	15.8	.64	.34
	93	41	21.8	.70	.53
	96	44	19.2	.81	.64
	106	48	23.2	.78	.64
	107	45	28.3	.76	.64
	111	45	22.5	.79	.66
	112	40	21.7	.75	.61
	114	37	19.5	.85	.74
	115	43	20.5	.79	.70
	116	45	17.8	.82	.70
	120	45	21.8	.66	.48
	121	39	16.8	.51	.36
	122	35	25.7	.50	.27
	125	44	19.7	.51	.32
	126	49	24.5	.89	.57
	127	33	14.2	.90	.80
	128	46	22.0	.50	.30
	131	48	23.0	.80	.54
	132	48	25.2	.62	.54

DESCRIPTION	I.D. NO.	HOLE NO.	DEPTH (ft.)	VISUAL	MC (initial)	LL	PL	PI	(C/P)nc	(C/P) AT OCR OF			
										2	5	10	20
Ft. Ann-Whitehall	1	21S	81.0	L,C;L & C	56	56.8	28.9	27.9	.26	.31	.47	.77	1.21
	2	142F	60.5	L & C	51	47.4	22.3	25.1	.19	.25	.62	.94	1.60
	3	111S	15.4	L & C	42	58.4	26.4	32.0	.53	.59	.78	1.18	1.67
	4	137S	20.5	L & C	45	63.2	24.2	39.0	.51	.71	1.33	2.40	4.28
	5	128S	80.5	L & C	56	58.2	25.4	32.8	:29				
	6	UH173	20.8	L,C,w/P	73	77.7	43.8	33.9	.32	.42	.73	1.33	1.82
	7	80F	20.5	L,C; L & C	44	41.8	20.5	21.3	.25	.36	.67	1.16	2.00
	8	80F	30.6	L,C; L & C	37	40.2	20.1	20.1	.23				
	9	UDH44F	5.4	O,L,C;L,C	30	43.5	20.6	22.9	.44	.57	1.00	1.60	2.40
	10	137S	80.9	L&C, L,C;LC	36	31.6	19.3	12.3	.24	.29	.45	.70	1.21
Port of Albany	11	UDH-X-2	34.3	LC,w/1yr CL	32	34.8	21.5	13.3	.61	.91	1.76	3.24	6.0
	12	UDH-X-2	41.1	LC; CL; SL	35	42.4	22.4	20.0	.42				
	13	UDH-X-2	47.2	CL, LC	28	25.1	15.9	9.2	.50	.74	1.54	2.86	7.5
	14	UDH-X-2	52.4	CL, LC	27	25.9	17.9	8.0	.59	.92	2.07	4.28	12.0
Whitehall Bypass	15	2A	40.4	L, C	43	44.1	26.2	17.9	.24				
Slingerland Byp.	16	UH-5	30.5	L, C	34	35.3	19.3	16.0	.38	.45	.61	.92	1.43
Int. 540-1-1	17	B13	45.8	L,C	31	33.3	16.2	17.1	.28				
	18	B13	25.6	L,C; LC	36	35.9	19.3	16.6	.32	.38	.56	.67	1.36
	19	B13	35.5	L,C; LC	36	33.5	16.0	17.5	.28				
Alt. Rt. 7	20	UDH-19	15.4	L,C; L & C	41	46.7	20.7	26.0	.36	.47	.82	1.36	2.86
	21	UDH-19	14.0	L,C	37	36.9	20.2	16.7	.30	.43	.87	1.71	3.75
	22	UDH-19	3.1	L,C,S, O	24	35.2	15.3	19.9	.64	.80	1.25	2.0	3.75
	23	UDH-19	14.0	L & C	37	36.9	20.2	16.7	.35	.51	1.07	1.94	3.33
	24	UDH-19	15.4	L,C, L & C	41	46.7	20.7	26.0	.36	.48	.87	1.54	2.86
Albany-Couse	25	UHMR-15	15.6	L & C, f.S	46	48.0	22.8	25.2	.29	.42	.91	1.40	3.75
	26	UHMR-15	25.7	L & C, f.S	49	43.0	20.6	22.4	.21	.37	.82	1.43	2.61
	27	UDTA15	20.4	L,C,S	34	29.4	18.4	11.0	.51	.57	.77	1.09	1.67
	28	UDTA15	5.3	L,C,S	30	55.1	26.1	29.0	.53	.72	1.35	1.94	3.16
	29	UDZ K	31.1	L & C, S	36	36.9	20.9	16.0	.34	.47	.84	1.54	2.50
	30	UDZ K	41.1	L & C	45	44.1	21.1	23.0	.26	.32	.53	.86	1.58
Alb. N' side Art.	31	UDH67	25.8	L & C, S	40	48.0	20.5	27.5	.53	.82	1.71	3.33	6.67
Castleton-Rens.	32	UD67	1.5	L & C	49	60.6	34.5	26.1	.81	1.25	2.31	3.75	6.0
	33	CW-3	26.7	C & L	–	40.0	19.2	20.8	.37	.58	1.25	2.31	4.28
	34	CW-3	15.4	LC, C & L	45	53.2	24.3	28.9	.40	.61	1.33	2.50	4.29
	35	UD68	9.7	L,C,S,O	41	48.1	24.4	23.7	.56	.65	1.05	1.82	3.16
	36	UD68	3.6	L & C	73	95.7	55.3	40.4	.76	.87	1.25	1.76	2.86

DESCRIPTION	I.D. NO.	HOLE NO.	DEPTH (ft.)	VISUAL	MC (initial)	LL	PL	PI	(C/P)nc	(C/P) AT OCR OF			
										2	5	10	20
South Mall	37	UH7-T4	15.6	L & C	45	50.8	27.3	23.5	.28	.40	.77	1.28	2.61
	38	UDH-1	25.4	L & C, S	36	41.4	22.0	19.4	.32	.48	.95	1.71	3.33
	39	UDH-1	35.6	L & C	36	33.4	19.7	13.7	.32	.50	1.00	1.67	3.75
	40	UDH3F	33.5	L, C, S	25	30.6	17.5	13.1	.47	.71	1.43	2.50	5.45
	41	UDH-?	35.5	L, C	31	30.8	17.5	13.3	.36	.61	1.20	2.07	4.00
	42	UDH-?	40.5	L, C	44	47.2	23.5	23.7	.32	.42	.77	1.30	2.61
	43	EH-1U	40.9		45	49.8	23.7	26.1	.33				
	44	EH-1U	60.8		49	55.3	26.6	20.7	.28				
	45	EH-1U	86.2		38	41.3	23.8	17.5	.36				
	46	EH-1U	116.0		30	33.5	22.9	10.6	.34				
	47	CC-2U	25.8		50	47.0	25.7	21.3	.42				
	48	CC-2U	50.8		32	32.3	18.6	13.7	.38				
	49	CC-2U	15.9		39	41.0	19.8	21.2	.48				
	50	PF-2U	45.9		40	40.7	22.3	18.4	.36				
	51	PF-2U	25.9		42	43.9	22.4	21.5	.35				
	52	AS-1U	50.8		47	54.7	26.5	28.2	.33				
	53	AS-1U	36.0		39	41.5	20.3	21.2	.31				
Rte 9W Ravena-Becker	54	UDH-1	21.0	L, C	57	58.9	25.9	33.2	.37	.54	1.02	1.76	3.53
	55	UDH-1	25.7	L, C.	36	32.6	16.2	16.4	.27	.33	.54	.84	1.62
	56	UDH-1	40.7	L,C, L-C	37	28.5	17.7	10.8	.24	.44	1.02	2.10	5.0
	57	UDH-1	50.7	L, C	44	43.1	22.8	20.3	.25	.41	.87	1.76	3.0
	58	UDH-1	80.7	L,C, L-C	38	34.8	20.2	14.6	.27	.36	.63	1.07	1.88
	59	UDH-1	62.7	L, C	42	37.4	22.4	15.0	.24	.51	1.13	2.50	7.50
Couse to Berksh.	60	UDH-95	21.1	L, C	33	20.7	15.2	5.5	.38				
	61	UDH-95	26.5	lay'd L, C	31	21.5	16.5	5.0	.38				

DESCRIPTION	I.D. NO.	HOLE NO.	DEPTH (ft.)	VISUAL	MC (initial)	LL	PL	PI	(C/P)nc	(C/P) AT OCR OF			
										2	5	10	20
Lockport Exp'y	62	537-A	29.0	C	48	52.1	25.5	26.6	.16	.27	.61	1.18	2.86
	63	534-A	29.4	C	46	46.3	24.5	21.8	.18	.27	.54	1.0	2.14
	64	534-A	27.6	C	47	49.0	26.6	22.6	.21	.33	.72	1.43	2.86
	65	534-A	22.3	C	46	51.2	25.0	26.2	.21	.24	.36	.91	2.0
	66	SI-418B	15.8	C w/lys L	42	39.7	20.4	19.3	.21	.30	.47	.78	1.71
	67	SI-418B	11.7	C w/poc L	43	44.6	22.5	22.1	.22				
	68	418-A	31.6	C	39	45.8	23.0	22.8	.24				
	69	418-A	33.5	C	42	48.8	23.0	25.8	.19				
	70	418-A	35.4	C w/L, G	40	42.3	21.5	20.8	.28	.33	.52	.67	1.0
	71	418-A	36.5	C w/occ lys L	41	44.1	20.8	23.3	.21	.30	.59	1.0	2.0
	72	418-A	40.3	C, pocs L	39	41.3	21.4	19.9	.18				
	73	416-A	34.4	C	44	44.9	23.5	21.4	.22				
	74	416-A	27.3	C w/lys L	39	50.4	25.3	25.1	.25				
	75	416-A	30.5	C w/pocs L	43	46.9	22.8	24.1	.24				
	76	416-A	33.6	C	39	43.9	22.7	21.2	.28				
	77	416-A	35.3	C, L	33	34.6	28.3	16.3	.21				
	78	416-A	37.0	C, pocs L, G	43	48.1	28.7	25.4	.22	.30	.44	.97	1.43
	79	416-A	41.2	C/lys L	21	18.2	12.1	6.1	.28				
	80	416-A	40.2	C/ly L	41	44.9	22.0	22.9	.18				
	81	416-A	31.9	C	40	47.0	24.4	22.6	.22				
	82	416-A	31.3	C	42	44.8	24.2	20.6	.11				
	83	416-A	32.7	C	39	43.8	22.8	21.0	.25	.39	.81	1.54	3.33
	84	244E	43.3	C w/pocs L, S	48	49.7	23.3	26.4	.18				
	85	244E	38.8	C w/lys ly S	40	48.9	23.6	25.3	.22	.32	.59	1.11	1.88
	86	244E	36.8	C w/pocs S	37	49.0	23.9	25.1	.28	.35	.57	.97	2.68
	87	244E	41.3	C w/lys L	45	49.0	24.9	24.1	.18	.24	.41	.73	1.36
	88	244E	40.4	layd C, lyc	38	39.8	21.4	18.4	.22	.32	.66	1.25	3.0
	89	244D	42.8	C, CL	36	39.0	20.8	18.2	.20	.26	.43	.73	1.28
	90	244D	44.6	C, CL	40	44.1	22.3	21.8	.19	.25	.36	.61	1.07
	91	244D	46.2	C w/pocs L, G	31	33.6	27.8	15.8	.22	.26	.40	.62	1.03
	92	244C	42.7	LC w/lys f.S	30	32.9	17.5	15.4	.29				
	93	244C	42.2	C, LC, CL	41	43.0	21.2	21.8	.19	.25	.46	.77	1.50
	94	244C	38.7	C w/lys L	32	41.0	21.5	19.5	.25				
	95	244C	40.4	C w/sm lys L	43	48.7	23.4	25.3	.17				
	96	244C	41.1	LC w/lys LS	44	39.8	20.6	19.2	.17	.26	.48	.91	2.0
	97	244B	23.8	C w/lys f.L S	42	47.7	21.4	26.3	.17				
	98	244B	20.4	C w/lys LC	41	46.2	22.1	24.1	.19				
	99	244B	22.2	C w/lys CL	44	49.2	23.6	25.6	.18				

DESCRIPTION	I.D. NO.	HOLE NO.	DEPTH (ft.)	VISUAL	MC (initial)	LL	PL	PI	(C/P)nc	(C/P) AT OCR OF			
										2	5	10	20
Lockport Exp'y (Cont.)	100	232B	12.7	C w/occ L	43	40.7	20.7	20.0	.18				
	101	232B	12.2	C w/lys C & L	48	52.0	23.9	28.1	.18				
	102	232B	10.4	C w/pocs L	36	53.2	24.8	28.4	.27				
	103	232B	7.0	C, lyr ly S	32	50.5	25.2	25.3	.22				
	104	232B	4.1	C, lyC,lyr LS	29	46.0	22.7	23.3	.24				
	105	232B	1.9	lyC w/lys L	28	45.5	22.7	23.8	.36				
	106	226A	36.2	C w/lys L	48	46.0	22.8	23.2	.18	.26	.53	.89	1.62
	107	226A	33.1	C w/thin lys L	45	53.8	22.5	28.3	.19	.28	.54	.94	1.82
	108	226A	22.5	C w/pocs L	38	46.0	22.8	23.2	.23				
	109	226A	37.6	C w/lys L	42	41.4	19.1	22.3	.19				
	110	226B	33.9	C w/lys L	43	44.0	23.1	20.9	.18				
	111	221C	37.2	C w/l lyr L	45	44.7	22.2	22.5	.17	.25	.54	.89	1.76
	112	221C	33.9	C w/lys L	40	43.7	22.0	21.7	.20	.29	.54	.97	2.14
	113	221C	35.6	C w/lys L	48	55.4	23.7	31.7	.14				
	114	551C	21.5	C w/L, O	37	40.4	20.9	19.5	.13	.21	.45	.82	1.62
	115	551B	12.6	C w/L	43	42.3	21.8	20.5	.23	.36	.75	1.40	2.31
	116	551B	14.1	C w/lys L	45	38.7	20.9	17.8	.21	.33	.67	1.25	2.40
	117	551B	10.8	C	42	48.0	23.5	24.5	.16				
	118	551B	15.9	C	51	48.3	24.5	23.8	.21				
	119	551A	24.9	C w/thin lys L	45	46.7	24.6	22.1	.20				
	120	551A	28.0	C w/lys LC	45	44.0	22.2	21.8	.19	.25	.44	.72	1.39
	121	551A	26.5	C w/lyr lyS	39	39.8	23.0	16.8	.21	.26	.38	.60	1.07
	122	534B	8.7	C	35	50.0	24.3	25.7	.29	.33	.47	.73	1.05
	123	534B	12.5	LC	45	45.9	25.2	20.7	.21				
	124	534B	15.6	LC	49	49.9	26.0	23.9	.21				
	125	534B	25.8	C w/lys L	44	42.8	23.1	19.7	.20	.24	.35	.62	.77
	126	534B	20.6	C	49	49.5	25.0	24.5	.22	.31	.57	1.05	1.94
	127	534B	27.4	lydCw/lyrsflyS	33	31.7	17.5	14.5	.22	.38	.80	1.62	3.0
	128	534B	22.4	C w/thin lys L	46	43.9	21.9	22.0	.23	.27	.38	.57	.83
	129	534B	14.4	C	47	51.3	24.9	26.4	.18				
	130	534B	10.4	C w/L lenses	41	47.1	23.4	23.7	.24				
	131	534B	17.3	C	48	48.6	25.6	23.0	.21	.29	.51	.86	1.76
	132	534B	19.0	C	48	49.3	24.1	25.2	.22	.29	.50	.79	1.50
	133	418B	22.5	C,lyC w/lyr L	31	30.3	15.3	15.0	.26				
	134	534A	50.9	C	46	50.9	25.8	25.1	.19				
	135	537A	28.4	C, lyC	51	52.5	25.5	27.0	.18				
	136	537A	21.6	C	45	50.4	25.9	24.5	.21				
	137	537A	23.4	C	47	51.2	21.0	30.2	.16				

DESCRIPTION	I.D. NO.	HOLE NO.	DEPTH (ft.)	VISUAL	MC (initial)	LL	PL	PI	(C/P)nc	(C/P) AT OCR OF			
										2	5	10	20
Lockport Exp'y (Cont.)	100	232B	12.7	C w/occ L	43	40.7	20.7	20.0	.18				
	101	232B	12.2	C w/lys C & L	48	52.0	23.9	28.1	.18				
	102	232B	10.4	C w/pocs L	36	53.2	24.8	28.4	.27				
	103	232B	7.0	C, lyr ly S	32	50.5	25.2	25.3	.22				
	104	232B	4.1	C, lyC,lyr LS	29	46.0	22.7	23.3	.24				
	105	232B	1.9	lyC w/lys L	28	45.5	22.7	23.8	.36				
	106	226A	36.2	C w/lys L	48	46.0	22.8	23.2	.18	.26	.53	.89	1.62
	107	226A	33.1	C w/thin lys L	45	53.8	22.5	28.3	.19	.28	.54	.94	1.82
	108	226A	22.5	C w/pocs L	38	46.0	22.8	23.2	.23				
	109	226A	37.6	C w/lys L	42	41.4	19.1	22.3	.19				
	110	226B	33.9	C w/lys L	43	44.0	23.1	20.9	.18				
	111	221C	37.2	C w/l lyr L	45	44.7	22.2	22.5	.17	.25	.54	.89	1.76
	112	221C	33.9	C w/lys L	40	43.7	22.0	21.7	.20	.29	.54	.97	2.14
	113	221C	35.6	C w/lys L	48	55.4	23.7	31.7	.14				
	114	551C	21.5	C w/L, O	37	40.4	20.9	19.5	.13	.21	.45	.82	1.62
	115	551B	12.6	C w/L	43	42.3	21.8	20.5	.23	.36	.75	1.40	2.31
	116	551B	14.1	C w/lys L	45	38.7	20.9	17.8	.21	.33	.67	1.25	2.40
	117	551B	10.8	C	42	48.0	23.5	24.5	.16				
	118	551B	15.9	C	51	48.3	24.5	23.8	.21				
	119	551A	24.9	C w/thin lys L	45	46.7	24.6	22.1	.20				
	120	551A	28.0	C w/lys LC	45	44.0	22.2	21.8	.19	.25	.44	.72	1.39
	121	551A	26.5	C w/lyr lyS	39	39.8	23.0	16.8	.21	.26	.38	.60	1.07
	122	534B	8.7	C	35	50.0	24.3	25.7	.29	.33	.47	.73	1.05
	123	534B	12.5	LC	45	45.9	25.2	20.7	.21				
	124	534B	15.6	LC	49	49.9	26.0	23.9	.21				
	125	534B	25.8	C w/lys L	44	42.8	23.1	19.7	.20	.24	.35	.62	.77
	126	534B	20.6	C	49	49.5	25.0	24.5	.22	.31	.57	1.05	1.94
	127	534B	27.4	lydCw/lyrsflyS	33	31.7	17.5	14.5	.22	.38	.80	1.62	3.0
	128	534B	22.4	C w/thin lys L	46	43.9	21.9	22.0	.23	.27	.38	.57	.83
	129	534B	14.4	C	47	51.3	24.9	26.4	.18				
	130	534B	10.4	C w/L lenses	41	47.1	23.4	23.7	.24				
	131	534B	17.3	C	48	48.6	25.6	23.0	.21	.29	.51	.86	1.76
	132	534B	19.0	C	48	49.3	24.1	25.2	.22	.29	.50	.79	1.50
	133	418B	22.5	C,lyC w/lyr L	31	30.3	15.3	15.0	.26				
	134	534A	50.9	C	46	50.9	25.8	25.1	.19				
	135	537A	28.4	C, lyC	51	52.5	25.5	27.0	.18				
	136	537A	21.6	C	45	50.4	25.9	24.5	.21				
	137	537A	23.4	C	47	51.2	21.0	30.2	.16				

TABLE NO. 2B-8

SUMMARY OF CONSOLIDATED UNDRAINED TRIAXIAL TEST RESULTS

NORMALIZED SOIL PROPERTIES COMPRESSION TESTS AT ALL SITES

	Sample Identification and Classification: Boring No	Sample No	Depth Below Mudline, FT	Elevation, FT	Soil Type	Natural Water Content, w_n, %	Liquid Limit, w_L, %	Plasticity Index, I_p, %	Existing Overburden Stress, $\bar{\sigma}_{vo}$, TSF	Laboratory Induced Stress History, At Maximum Stress: Vertical Effective Stress, $\bar{\sigma}_{vmax}$, TSF	Volumetric Strain, ε_v, %	At Start of Shear: Vertical Effective Stress, $\bar{\sigma}_{vc}$, TSF	Volumetric Strain, ε_v, %	Water Content at End of Test, w_f, %	Coefficient of Lateral Pressure, $K = \bar{\sigma}_{hc}/\bar{\sigma}_{vc}$	Overconsolidation Ratio, OCR, $\bar{\sigma}_{vmax}/\bar{\sigma}_{vc}$	At Maximum Stress Difference: Compressive Strength $(\sigma_1-\sigma_3)$, TSF	Normalized Shear Strength $S_u/\bar{\sigma}_{vc}$ (C/P)	Axial Strain, ε, %	Pore Pressure Coefficient, A	Average Principal Stress, $\bar{p} = \frac{1}{2}(\bar{\sigma}_1+\bar{\sigma}_3)$, TSF	Principal Stress Ratio, $\bar{\sigma}_1/\bar{\sigma}_3$	At Maximum Stress Ratio: Principal Stress Ratio, $\bar{\sigma}_1/\bar{\sigma}_3$	Axial Strain, ε, %	Pore Pressure Coefficient, A	Average Stress Difference, $q = \frac{1}{2}(\sigma_1-\sigma_3)$, TSF	Average Principal Stress, $\bar{p} = \frac{1}{2}(\bar{\sigma}_1+\bar{\sigma}_3)$, TSF	Initial Tangent Modulus Ratio, E_i/S_u
						w_n	w_L	I_p	$\bar{\sigma}_{vo}$	$\bar{\sigma}_{vmax}$	ε_v	$\bar{\sigma}_{vc}$	ε_v	w_f	K	OCR	$(\sigma_1-\sigma_3)$	$S_u/\bar{\sigma}_{vc}$	ε	A	$\bar{p}$	$\bar{\sigma}_1/\bar{\sigma}_3$	$\bar{\sigma}_1/\bar{\sigma}_3$	ε	A	q	$\bar{p}$	E_i/S_u
	CIU-C TESTS																											
138	RR-116U	22U-B	68.2	-83.7	OH-OH	61.9	61	31	1.07	2.93	17.6	2.93	17.6	44.6	1.0	1.0	1.97	0.34	5.1	1.06	1.77	3.49	4.08	15.5	1.24	0.91	1.50	784
139		22U-D	68.8	-84.3	OH-OH	61.9	61	31	1.07	2.91	16.9	1.70	16.5	45.5	1.0	1.7	1.85	0.54	5.5	0.56	1.59	3.79	4.37	15.5	0.67	0.87	1.39	749
140		22U-A	67.8	-83.3	OH-OH	64.5	61	31	1.07	2.94	18.5	0.94	17.2	47.1	1.0	3.1	1.84	0.98	12.3	0.25	1.43	4.62	4.62	12.3	0.25	0.92	1.43	302
141	RR-119U	4U-A	13.1	-26.6	OH	103.3	91	47	0.12	0.70	26.2	0.70	26.2	67.5	1.0	1.0	0.53	0.38	19.4	1.00	0.37	6.26	6.37	21.2	1.02	0.26	0.36	375
142		4U-B	13.6	-27.1	OH	101.3	91	47	0.12	0.71	26.6	0.38	26.1	67.5	1.0	1.9	0.50	0.64	14.4	0.48	0.40	4.28	4.50	19.2	0.51	0.24	0.36	509
143		4U-D	14.2	-27.7	OH	98.8	91	47	0.12	0.71	22.7	0.20	21.1	71.1	1.0	3.6	0.46	1.18	14.2	0.25	0.34	5.31	5.37	18.1	0.27	0.22	0.33	256
144	RR-119U	16U-D	54.9	-68.4	OH	58.8	59	27	0.82	2.44	15.6	2.44	15.6	43.5	1.0	1.0	1.60	0.33	6.3	1.12	1.39	3.74	4.10	13.9	1.21	0.76	1.27	1024
145		16U-E	55.3	-68.8	OH	58.0	59	27	0.82	2.44	15.6	1.42	15.3	43.5	1.0	1.7	1.48	0.52	3.6	0.64	1.35	3.40	3.99	13.9	0.66	0.71	1.10	964
146		16U-C	54.5	-68.0	OH	59.2	59	27	0.82	2.44	16.4	0.59	15.0	44.8	1.0	4.2	1.34	1.14	10.3	0.16	1.08	4.33	4.34	12.0	0.16	0.67	1.07	304
147	RR-131U	11U-D	50.7	-64.7	OH	57.3	53	23	0.84	1.95	16.8	1.95	16.8	43.3	1.0	1.0	1.55	0.40	14.8	0.92	1.26	4.23	4.23	13.7	0.92	0.78	1.26	582
148		11U-B	49.8	-63.8	OH	54.6	53	23	0.84	1.95	16.7	0.95	16.2	43.9	1.0	2.1	1.41	0.75	15.6	0.35	1.18	3.98	4.03	12.4	0.36	0.70	1.16	351
149		11U-C	50.3	-64.3	OH	59.1	53	23	0.84	1.95	15.9	0.46	14.5	44.7	1.0	4.2	1.20	1.31	13.6	0.08	1.01	3.97	4.03	10.2	0.10	0.59	0.98	106
150	RR-131U	21U-D	74.6	-88.6	OH-OH	63.0	72	38	1.38	2.93	16.4	2.93	16.4	48.0	1.0	1.0	2.08	0.35	4.9	1.04	1.87	3.51	4.22	14.2	1.16	0.57	1.58	675
151		21U-C	74.2	-88.2	OH-OH	58.1	72	38	1.36	2.94	14.5	1.71	14.1	45.4	1.0	1.7	1.94	0.57	4.6	0.50	1.70	3.65	4.16	12.4	0.62	0.91	1.49	743
152		21U-E	73.8	-87.8	OH-OH	55.6	72	38	1.38	2.94	14.5	0.95	13.4	43.9	1.0	3.1	1.67	0.88	4.9	0.23	1.42	3.86	4.14	12.0	0.27	0.82	1.34	416
153	RR-132U	31U-E	135.7	-176.7	CH	53.8	58	29	3.13	4.93	15.8	4.93	15.8	40.3	1.0	1.0	3.05	0.31	3.2	1.11	3.01	3.06	3.91	13.8	1.41	1.38	2.33	948
154		31U-C	140.1	-177.1	CH	43.1	58	29	3.13	4.95	12.7	4.95	12.7	35.4	1.0	1.0	3.14	0.32	2.8	1.10	3.01	3.18	3.89	11.7	1.34	1.45	2.45	1207
155	WPT-108U	14U-B	39.8	-68.6	OH-OH	51.9	55	28	0.65	2.15	12.2	2.15	12.2	41.2	1.0	1.0	1.46	0.34	7.8	1.07	1.27	3.73	3.84	12.8	1.11	0.72	1.22	1097
156		14U-D	40.6	-69.4	OH-OH	54.3	55	28	0.65	2.15	13.1	1.24	12.5	43.0	1.0	1.7	1.30	0.52	4.9	0.63	1.20	3.38	3.71	11.1	0.60	0.63	1.10	1240
157		14U-E	41.0	-69.8	OH-OH	49.0	55	28	0.65	2.16	13.0	0.69	12.0	38.8	1.0	3.2	1.21	0.89	9.1	0.22	1.04	3.78	3.78	10.6	0.22	0.60	1.04	585
158	WPT-109U	15U-B	37.9	-53.6	OH	53.8	62	30	0.64	2.13	18.3	2.13	18.3	37.4	1.0	1.0	2.20	0.52	5.9	0.63	1.83	4.01	4.10	9.8	0.66	1.08	1.77	
159		15U-C	38.7	-54.4	OH	62.0	62	30	0.64	2.14	17.8	1.04	17.3	45.0	1.0	2.1	1.32	0.64	7.9	0.46	1.10	4.02	4.18	13.4	0.50	0.64	1.04	899
160		15U-C	38.3	-54.0	OH	60.3	62	30	0.64	2.14	17.7	0.40	15.9	45.1	1.0	5.3	1.05	1.30	11.7	0.10	0.86	4.16	4.16	11.0	0.10	0.53	0.86	379
161	WPT-113U	4U-A	19.1	-49.6	OH	131.6	129	78	0.20	0.87	34.0	0.87	34.0	78.5	1.0	1.0	0.69	0.40	15.8	0.96	0.48	6.01	6.35	19.7	0.98	0.34	0.47	313
162		4U-C	19.8	-50.3	OH	142.7	129	78	0.20	0.87	34.9	0.46	34.3	82.4	1.0	1.9	0.60	0.65	13.7	0.56	0.42	6.01	6.69	18.5	0.55	0.30	0.40	455
163		4U-D	20.3	-50.8	OH	148.4	129	78	0.20	0.66	37.2	0.20	35.6	84.7	1.0	4.3	0.48	1.20	14.2	0.25	0.36	5.23	5.62	15.6	0.26	0.24	0.34	212
164	WT-124U	14U-B	41.3	-61.8	CL-CL	39.3	40	18	0.64	2.72	14.1	2.72	14.1	28.7	1.0	1.0	1.71	0.31	5.5	1.16	1.55	3.46	3.63	10.7	1.21	0.84	1.47	1480
165		14U-D	42.2	-62.7	CL-CL	45.7	40	18	0.64	2.73	19.6	0.63	18.4	31.2	1.0	4.3	1.45	1.14	12.6	0.09	1.27	3.69	3.69	11.9	0.09	0.72	1.26	532
166	WT-146U	9U-D	37.3	-56.5	CL-CL	47.5	49	25	0.49	1.95	15.8	1.95	15.8	36.6	1.0	1.0	1.30	0.33	19.8	1.09	1.12	3.74	3.76	17.4	1.10	0.65	1.10	717
167		9U-B	36.4	-55.6	CL-CL	49.4	49	25	0.49	1.95	15.1	0.95	14.7	35.9	1.0	2.0	1.26	0.66	6.0	0.37	1.12	3.55	3.69	12.1	0.42	0.61	1.06	1223
168		9U-C	36.8	-56.0	CL-CL	49.5	49	25	0.49	1.95	14.2	0.85	12.9	38.7	1.0	4.4	1.03	1.15	6.6	0.11	0.90	3.71	3.78	12.1	0.13	0.50	0.67	452
	CAU-C TESTS																											
169	RR-116U	30U-D	34.1	-49.6	OH	91.6	86	47	0.37	2.40	30.9	2.40	30.9	53.2	0.48	1.0	1.79	0.37	1.0	0.64	1.70	3.22	4.35	86.2	2.84	0.75	1.20	924
170		30U-F	33.4	-48.9	OH	85.8	86	47	0.37	1.60	23.4	0.94	23.1	58.1	0.60	1.7	1.14	0.60	3.0	0.23	0.96	3.93	4.56	35.0	0.42	0.52	0.81	1293
171		30U-A	33.0	-46.5	OH	90.9	86	47	0.37	1.60	26.3	0.52	25.4	59.5	0.74	3.1	1.03	0.99	6.3	0.12	0.79	4.75	4.97	33.2	0.16	0.50	0.75	746
172	WPT-1..U	25U-B	43.0	-71.8	OH-OH	50.3	58	29	0.72	2.18	9.4	2.18	9.4	42.3	0.48	1.0	1.54	0.35	1.5	0.72	1.53	3.03	3.54	25.8	2.43	0.67	1.20	534
173		25U-C	43.4	-72.2	OH-OH	52.7	58	29	0.72	2.21	10.6	1.30	10.2	43.6	0.60	1.7	1.45	0.56	1.9	0.22	1.31	3.50	3.73	8.7	0.40	0.65	1.12	1573
174		25U-D	43.9	-72.7	OH-OH	53.4	58	29	0.72	2.21	10.1	0.69	9.2	45.4	0.81	3.2	1.25	0.90	3.9	0.09	1.09	3.70	3.73	1.7	0.12	0.59	1.03	534
	CK_0U-C TESTS																											
175		4U-A	15.1	-30.6	OH	95.7	99	54	0.21	0.63	19.2	0.21	18.5	72.7	0.79	3.0	0.46	1.10	2.6	0.09	0.36	4.58	6.05	11.3	0.15	0.22	0.32	548
176		11U-B	54.8	-68.8	OH	61.1	63	29	0.94	1.99	11.5	1.99	11.5	49.8	0.46	1.0	1.47	0.37	0.9	0.71	1.37	3.30	4.25	9.6	2.70	0.63	1.03	785

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